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The Twin Falls-Oakley Irrigation Project

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THE TWIN FALLS-OAKLEY IRRIGATION PROJECT

BY

AMUND MARIUS KORSMO

B. S. University of Illinois, 1909

THESIS

Submitted in Partial Fulfillment of the Requirements for the

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April 3, 1913.

I hereby recommend that the thesis prepared by AMUND MARIUS KORSMO, A.B., B. S., entitled THE TWIN FALLS-OAKLEY IRRIGATION PROJECT, be accepted as fulfilling this part of the requirements for the degree of Civil Engineer.

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Recommendation concurred in:

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246493

PREFACE.

The thesis submitted herewith, was written with the object of describing the reconnoisance and design of irrigation works in general, as well as treating of the more particular conditions involved in the promotion of the Twin Falls-Oakley Project, ~~and~~ To this, is added adding a chronological record of the construction methods employed in the development of this ~~one individual~~ irrigation scheme. The writer fully realizes that the subject chosen is too large to develop thoroughly within the limits of a thesis of 25,000 or even 50,000 words, but considering the time available and other conditions under which it was written, no other course was possible. Being located in a construction camp, without access to an engineering library, no considerable amount of data was available on any one feature of irrigation, such as design of earth dams, design of distribution systems, etc., from which a more valuable thesis would have resulted.

The writer wishes to acknowledge the receipt of data on design and construction of irrigation works from the following sources; Mr. R. W. Faris, for permission to write a thesis on the Oakley Project, also for access to the preliminary notes, estimates and designs; Mr. A. J. Wiley, Consulting Engr. and Mr. H. Conkling, Asst. Engr., for data on designs of distribution systems of the U. S. Reclamation Service; Mr. W. C. Locke, Engr. Centerville Mill & Mining Co., Idaho, for data on slucing earth; Mr. E. L. Wells, Director Weather Bureau, Boise, Idaho, for data on evaporation losses in reservoirs; Mr. J.M. Burkett,

Carey Act Engr., Idaho, for data on transportation losses in earth canals, Mr. R. C. Pierce, Engr. U. S. Geological Survey for data on stream measurements, and Mr. C. A. McClelland, Hydrographer, Twin Falls North Side Project, Idaho, for data on the coefficient "N" for earth canals.

The following text books were also consulted; Wilson's, Irrigation Engineering, and Newell's, Irrigation in the United States.

A. M. K.

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THE DESERT.

THE DESERT RECLAIMED.

THE TWIN FALLS-OAKLEY IRRIGATION PROJECT.

INTRODUCTION.

Irrigation, as the word is used today, means the watering of land by artificial methods, supplementing the natural rainfall. In its wider sense, irrigation includes the whole question of conservation and utilization of water in the development of arid regions. Its origin dates back in the ancient history of India and Egypt, ~~and~~ in our own country, the Indians and Cliff Dwellers of the Southwest practised their crude methods long before the coming of the white man.

In the United States irrigation is used almost wholly in the region west of the 97th Meridian. This territory is classed as arid, in other words, as having a precipitation of 20 inches or less annually. Irrigation is by no means confined to arid regions; but rather to those where the rainfall is not sufficient during the growing season to develop crops.

There still exists the mistaken idea that irrigation is a hardship, and its necessity a misfortune. Nothing could be more misleading, for the opposite is more nearly true. A lack of precipitation in arid regions implies a dry, warm climate, plenty of sunshine; and in the event that an irrigation project is feasible, means a regular and proper watering of crops, which is not always possible when the natural rainfall is relied upon.

1. KINDS OF IRRIGATION.

In general, there are three kinds or classes of irrigation. First, the Gravity System, when the water is brought on the land through ditches, by the aid of gravity. Of this class there are two divisions, one, where the water used is diverted to the canals by means of a dam across a perennial stream, and the other, when a storage dam is necessary to impound water during the winter months and discharge it to the canals during the irrigating season.

The second class is the Pumping System, where the land is at a higher elevation than the water with which it is to be irrigated. Hence, pumps are installed and the water lifted to the high point of the land from whence it is distributed by gravity ditches or pipes.

The third class is the Artesian Well System. Here, the water rises up out of the ground by its own energy and is distributed on the land by gravity ditches or pipes.

There are other systems that employ combinations of these three classes, chief among which is that of Sub-Irrigation. Here the water is brought on the land by canals and then held in check by means of dykes thrown across them. The water percolates into the land, and because of an impervious strata some 5 to 25 feet below the surface, it penetrates no further, but spreads laterally, finally coming to the surface and irrigating the adjacent land. When the soil has had sufficient water, the dykes are opened and the canal drained.

This operation is then repeated on the next section to be irrigated. Naturally, this system is only adapted to favorable geological conditions. It has been used with great success in the Snake river valley at St. Anthony, Idaho.

Water for irrigation may differ from that used for domestic purposes in that the former need not be pure, in fact, considerable irrigation is carried on by using the sewage of large cities.

Irrigation by pumping is limited, because of its operating cost, to a maximum lift of 50 to 80 feet. Installation, usually electric, costs from $5.00 to $7.50 per acre, and operation from $1.75 to $2.25 per acre. The average cost of gravity systems in the United States is about $15.00 per acre, at the present time.

Surface water supplied 90% to 95% of the irrigation works used in 1910. Of the remainder, 3% was Pumping and 1% Artesian Well.

The following table shows the amount of land irrigated in the principal countries in 1909.

United States	14,000,000	Acres
India	40,000,000	"
Egypt	6,000,000	"
Italy	4,700,000	"
Spain	2,800,000	"
France	400,000	"

Besides the 14,000,000 acres irrigated in the United States in 1909, 5,000,000 more acres could have been irrigated under the same systems. In 1910, 31,000,000 acres were capable of irrigation under the systems completed or under construction, and beyond a doubt the United States will lead all other countries in 1915 for total lands irrigated.

2. CLASSIFICATION OF WORKS.

In the classification of irrigation works, as regards their promotion and development, three important divisions occur, namely, the United States Reclamation Service, the Carey Act, and the private individual or corporation.

One third of the United States, exclusive of Alaska and outlying possessions, are vacant public lands. Furthermore, the arid regions of the United States include more than two-fifths of its entire area. These facts, together with the report of the Geological Survey in 1901, which stated, after a thorough investigation of conditions in the arid west, that less than one per cent of the total land area was irrigated, also that these tracts were surrounded by others 5 to 10 times as large which were capable of Irrigation under ideal conditions, caused the Reclamation Act to be passed June 1902, the Carey Act having become a law in Aug. 1894.

Briefly stated, the Reclamation Act operates as follows. The government, with the money accruing from the sale of public lands, designs and builds irrigation projects, reclaiming desert lands. The land is sold to the public in units fixed by the Secretary of the Interior, and vary from 40 to 160 acres in size. The selling price is based on the cost of the irrigating system, and is assessed proportionally among the settlers. In the past this has varied from $22.00 to $60.00 per acre. The entryman is required to live on his land for 3 years and reclaim at least one-half of it for

agricultural purposes, before he can obtain title to it. He has ten years in which to pay for his land, at the end of which period, the community, organizing a Water Users Association, takes over and operates the system, having then become sole owners of it.

The Carey Act operates somewhat differently. In the first place the Federal Government deeds over to the several States, public lands in units of one million acres. Private capital may then design and construct irrigation works to reclaim these desert lands. The farm unit is fixed at a maximum of 160 acres, the State receiving 50¢ per acre for every acre in the segregation, and at the same time, fixing the rate or price at which the land is to be sold by the promoters. This rate has varied from $30.00 to $65.00 per acre in Idaho. The entry man is required to live on his land 30 days, and to reclaim one-eighth of it for agricultural purposes within the first 3 years. When this is done, he receives final title to it. Ten years are allowed the settlers in which to pay for their land, and after 3 year's operation by the promoters, the system is turned over to the farmers who then organize a Water Users Association similar to the U. S. R. S.

In the same way, homesteaders may group together, furnish their own capital and build a private system. The state, however, maintains an active supervision over the construction of all systems, not only private but governmental, as well.

A comparison between the Reclamation and Carey Act projects, as regards superior advantages is difficult, because

of the fact that each serves a different type of settler. The U..S. R. S. entry man receives his land and water at cost but is required to live on it much longer than his Carey Act neighbor, while the latter must pay for the profit demanded by the promoter, but gets a clear title to his land in a short time, upon which he may borrow additional capital.

Another phase of the Carey Act which is not generally appreciated as yet, is the way it evades true conservation, at times. Often segregations are opened that do not include all the land available, simply because the cost of such construction would reduce the profits of the promoters. The larger tract is then condemned by the smaller one. With the U. S. R. S., profit does not enter into the consideration of designs of projects, the latter being limited only by their feasibility.

Private and cooperative enterprises lead all others in the amount of acreage irrigated, having in 1909 - 94% of the total, leaving 3% for the U. S. R. S., 2% for the Carey Act, and 1% for the Indian Service. In 1910, however, the Carey Act passed the U. S. R. S., having 8% against 6% for the latter, and 3% for the Indian Service.

3. IRRIGATION STATISTICS.

On page 9 is given a summary, issued by the Department of Commerce and Labor in 1909, showing the extent of irrigation development in the United States. This remarkable advancement is an increase of 55% in the last ten years. Another fact brought out by the table is the increased cost per acre of building irrigation systems. However, while this increase amounts to 77%, the corresponding increase in the land values has reached 185%. It is but natural that the cost of systems should increase, for those built in the past were cheaply and poorly constructed, and the same reason explains the increased cost of maint~~ai~~enance for the last 10 years.

SUMMARY OF CENSUS FIGURES ON IRRIGATION

IN ARID AND SEMI-ARID REGIONS OF THE UNITED STATES.

Description	1909	1899
Number of farms	1,439,023	1,095,675
Approximate land Area, Acres.	1,161,385,600	1,161,385,600
Improved land in farms, Acres	173,433,209	119,709,592
Av. Value per acre of farm land	$ 26.99	$ 9.42
Acres Irrigated	13,739,499	7,527,690
Area enterprises were capable of supplying in 1910	19,335,711	
Area included in projects, 1910	31,112,110	
Percent of total land area irrigated	1.2	0.6
Acreage irrigated by pumped water.	478,288	
Acreage irrigated by flowing wells.	125,590	
Total cost of irrigating systems.	$ 304,699,450	$ 237,217,189
Av. cost per acre.	$ 15.76	$ 8.89
Av. annual cost of maintenance and operation.	$ 1.07	$ .38

The Twin Falls-Oakley Project

PRELIMINARY

1. RECONNOISANCE.

The Twin Falls-Oakley Project comes under the Carey-Act type of enterprise. It is financed by the Kuhn Bros., of Pittsburgh; and the Faris Engineering and Construction Company of Boise, Idaho, are the Engineers, Contractors, and Builders. The project is located in the Goose Creek Valley, of Cassia Co., Idaho, about 40 miles east of Twin Falls. Preliminary work was begun ~~here~~ in the spring of 1907.

The valley is 25 miles long, 20 miles wide at its lower end, and tapers to an apex at the upper end, where it enters the canyon. High mountain ranges surround it on all sides except the north, where the Snake river valley is its natural outlet.

Four miles from the upper end of the valley lies the town of Oakley, which is inhabited by a small Mormon community. In 1907 about 6000 acres of land were under cultivation in that immediate vicinity; but as the town was 22 miles from the railroad, agriculture was not carried on any commercial scale, but merely to supply the local demands. Water for irrigation was taken from a number of creeks which flowed into the valley, namely, Goose, Trapper, Cottonwood, Birch and Basin Creeks. A crude irrigation system of canals

had been built, but no provision was made to impound the flood waters in the spring or the natural flow outside the irrigating season. As a result, water bacame very scarce during the growing season, especially for those who had second and third water rights. These creeks are perennial streams and like all others in mountainous countries vary greatly in their flow during the different seasons. Goose creek, for example, ranges from 35-40 sec. ft. in the summer months to 1000-1500 sec. ft. during the flood season. There were ~~also~~ very few wells in the valley, water for domestic purposes coming from these creeks, which are fed by melting snows on the mountains.

The valley has a gentle slope to the northeast and is exceptionally flat. The desert land is covered with a heavy growth of sage brush and shad scale, and ~~this being~~ the home of thousands of jackrabbits, scorpions, and other forms of desert life.

The first summer was spent largely in taking stream measurements, making examinations of the soil, obtaining data on water supply, precipitation, water duty, etc., from the farmers in the community. Preliminary surveys were also made on the proposed distribution system from which an estimate of design could be made.

2. CONDITIONS FOR FEASIBILITY OF PROJECT.

(a) WATER SUPPLY.

There are four conditions which govern the feasibility of all irrigation projects, namely, (1) the water supply, (2) the duty of the water, (3) the land, and (4) the cost.

Here, the source of water supply is the melted snows on the drainage area tributary to the four creeks, vis., Goose, Trapper, Birch, and Cottonwood. This region lies on the northern slope of the Columbia River Drainage Basin, located in Utah, Idaho and Nevada. Its average elevation is 6500 ft., some of the higher peaks towering 11,000 ft. above sea level. The size of the drainage area had to be obtained by actual survey, ~~of the water-shed~~, there having been no ~~previous~~ ~~government~~ survey~~s~~. There are 870 square miles in the ~~above~~ drainage area.

Unfortunately, no records of runoff had been kept in the valley previous to the reconnoisance surveys, and through an accident the measuring weir, built in 1907 for that purpose, was destroyed. The only alternative then, to determine the runoff was by means of precipitation and run-off formulae, ~~also~~ a comparison with other tracts, subject to similar conditions. Considering the importance of such a matter as the water supply for an irrigation project, this course is not desirable, being to a certain extent unreliable, and tending to give an average runoff, rather than the minimum, which should govern in this consideration. However, to offset any great errors which might result from the use

of this method, there was the evidence of the settlers in the valley to either confirm or disprove the conclusions arrived at.

Below is a table prepared from U. S. Weather Bureau observations which shows the precipitation at Oakley and stations in its vicinity.

PRECIPITATION.

Station.	Period.	Elevation.	Av. Precipitation.
Oakley	15 yrs.	4700 ft.	9.22 inches.
Albion	7 "	5500 "	13.64 "
Standrod	2 "	6000 "	19.06 "
American Falls	15 "	4330 "	13.19 "
San Jacinto	3 "	5000 "	11.72 "
Milner	4 "	4210 "	12.57 "
Twin Falls	3 "	3800 "	15.03 "
Pocatello	33 "	4466 "	12.93 "
Hot Springs	2 "	3000 "	10.47 "
Boise	33 "	2760 "	12.71 "
	Average	4377 "	13.05 "

It is a well known fact that precipitation increases with elevation, but this rate usually is a variable. Mr. F. H. Newell of the Reclamation Service has a rule that precipitation increases 0.6 of an inch per annum for every 100 ft. rise in elevation. This has been found to check very closely in the Sierras of California and upon investigating the data in the above table for Oakley, Albion and Standrod, three towns in

the drainage area under consideration, it is also found to check very well.

As stated before, the average elevation of the drainage area is 6500 ft. or 1800 ft. above Oakley. Applying Mr. Newell's rule gives 10.8 inches increase in precipitation or 20.02 inches as an average for the entire area. This estimate of 20 inches is considered conservative, first, because both the Weather Bureau and the residents of Oakley are positive that 9.2 inches is too low a figure for precipitation at that place, and second, on account of the favorable location of the drainage area on the north slope of the Columbia Basin.

The following table gives the annual run off for nine drainage basins in the arid west.

RUNOFFS FROM VARIOUS DRAINAGE BASINS.

Basin.	Period.	Area	Elev.	Av.Precipitation.	Runoff per Sq.Mi.
Ashley Cr.U.	1901-05	250 Sq.Mi.			368 A.Ft.
Bear Riv.U	1903-04	6000	5500		415
Boise Riv.Ida.	1901-04	2450	6000	14.42 In.	771
Blackfoot Ida.	1905-06	1016	6500	17.51	175
Spanish Fork U.		670		14.78	121
Brunean Riv.Ida.	1895-03	1800	4500		167
Salt Riv. Ariz.		6260		22.82	179
Sweetwater Cal.		186	2800	7.00	83
Weiser Riv.Ida.	1901-04	1670	3000		
		Average			284

The determination of run off is even more difficult than ascertaining the precipitation. It varies greatly according to the rate of precipitation, the steepness of the drainage area, and also with the amount of pervious material covering the slopes.

Mr. Grumsky, late of the U. S. Geological Survey, after a long study of run-offs and precipitation, established the following relation between the two. Precipitation, in inches, expresses numerically the per cent of runoff, or in other words, if the precipitation were 25 inches annually, the runoff would be 25% of that amount.

It can easily be seen how this rule may be subject to error when all the above factors of runoff are considered. There are, however, a great number of cases where this rule has proven quite accurate, notably at the Croton Aqueduct in New York.

Taking the precipitation of the drainage basin as 20 inches over 870 square miles, gives 928,000 acre feet, and applying Mr. Grumsky's rule of 20%, gives 185,600, or 213 acre feet per square mile. As compared to the above table it is found to be 25% less than the average there. Existing records of runoffs from 38 other catchment basins in the west show that the average amount furnished for storage is 540 acre feet per sq. mile of drainage area. Again the estimate made on the Oakley basin falls below the average and should be considered safe.

The most complete records available of actual run-offs in this vicinity are those of the Brunean River, Idaho.

Its drainage area is approximately 1800 square miles at an average elevation of 4500 ft. U. S. G. S. records, covering a period of 9 years, show an annual run off of approximately 300,000 acre ft. or 167 per square mile of drainage area.

At a measuring station some 20 miles away, the elevation is about 3000 ft. and the average precipitation is 10.5 inches. Using Mr. Newell's rule to determine the increased precipitation on the drainage basin, gives 19 inches. Now, applying Mr. Grumsky's rule of 19% for run off, gives 3.6 inches, which on an 1800 sq. mile drainage area would amount to 345,000 acre ft., thereby checking the actual run off quite closely. The Brunean Basin is lower than that of Goose Creek, hence may be expected to have less precipitation, and also, as it has a much more gentle slope, should naturally have less run off. But for ultra conservatism, the run off on the Goose Creek Basin was based on the above records at Brunean River, and at 167 acre ft. per sq. mi. gave 148,420 acre ft. from which storage and transportation losses were to be deducted.

As stated above, this method of deducing the available water supply is undoubtedly more or less arbitrary and unsatisfactory, nevertheless, the fact that all settlers, after knowing the scope and extent of the proposed project, were willing to surrender their water rights and sign new contracts the same as incoming entry men, surely is proof that the above estimated water supply is available.

(b) DUTY OF WATER.

In irrigation, the duty of water means the ratio between a quantity of water and the area of land on which it will mature a crop. This relation may be expressed in several different ways, the most common being the amount of water necessary to irrigate one acre of land. Some Engineers include in this the amount lost by evaporation and seepage while in transportation, and other base water duty on the actual amount delivered to the land. Consequently, the term, duty of water, is rather indefinite. The unit of water used in this notation is the acre foot, or an acre (43560 sq. ft.) one foot deep.

Because of the indefinite manner of expressing water duty, data from other tracts, usually is of little value. It is a well known fact that the U. S. R. S. have been very liberal with their water, allowing from 2 to 3½ acre ft. actually delivered to each acre. In several cases they have regretted this later, the land becoming so sour and swampy that extensive drainage systems had to be installed.

In the Oakley valley everything tends toward a high duty of water. The land is very fertile, the soil is deep, and has a gentle, even slope with an ideal under-drainage of gravel. The variation from too little to too much water for irrigation is not a large one and requires an experienced hand to get the best results. As a usual thing, water is being wasted on most projects and thus eventually carries with it a heavy penalty in the shape of swampy lands, if such methods are continued for some time. Potatoes in this valley,

for example, will grow and mature much better with one irrigation than with three. Fall plowing is also an important factor, as the moisture absorbed during the winter and spring months will usually take the place of one irrigation.

As a rule, 500 out of every 640 acres are never irrigated, being occupied by barns, roads, fences and houses. For a conservative estimate of water duty, this fact should not be considered, rather allowing it as a safety factor in the final fixed duty.

When questioned regarding the duty of water in this valley, the local settlers were unanimously of the opinion that 1.5 acre ft.were entirely sufficient to irrigate any of their crops. Government reports, issued by the Irrigation Experiment Station, substantiate the experience of the farmers at Oakley. Below are shown tables of several experiments, conducted by the U. S. Department of Agriculture on the duty of water in the Gallotin Valley, Montana, showing the √ proceedure followed and results obtained. These experiments were made on grains and grasses but a great many others have been made on fruits and vegetables, and only in a very few instances were more than 1.5 acre ft. required to mature crops for maximum returns.

EXPERIMENT NO. 1.

Crop - Clover, 27.44 Acres.

	First Irrigation	Second Irrigation	Total.
Date of Irrigation	June 17-22	July 26-Aug.2	
Duration of Irrigation, Hrs.	104	115	219
Area Irrigated, Acres	27.44	27.44	27.44
Water Used, Acre Ft.	13.55	14.51	27.92
Depth Applied, Ft.	.49	.53	1.02
Head Used, Sec. Ft.	1.55	1.53	

Yield, 3.28 Tons per Acre.

Soil 6" vegetable loam, 20 inches clam loam, 40 inches clay marl over unknown depth of gravel and cobble stones. Practically no water wasted.

EXPERIMENT NO. 3.

Crop, Barley, 5.25 Acres - Wheat, 6.0 Acres

	First Irrigation	Second Irrigation	Total
Date of Irrigation	June 23-27	July 12-14	
Duration of Irrigation, Hrs.	96	56	152
Area Irrigated, Acres	11.27	11.27	11.27
Water Used, Acre Ft.	14.34	7.96	22.30
Depth Applied, Ft.	1.27	.71	1.98
Head Used, Sec. Ft.	1.81	1.72	

Yield, Wheat, 57 Bu. per A. - Barley, 45 Bu. per A.

Soil somewhat richer than that described in Experiment No. 1. More vegetable mould on top, 6 ft. of gravel, steep ground, water allowed to run all night. Undoubtedly considerable waste.

EXPERIMENT NO. 4.

Crop, Barley and Oats --- 66.39 Acres.

Date of Irrigation	July 5-13
Duration of Irrigation	196 Hours
Area Irrigated	66.39 Acres
Water Used	65.37 Acre Ft.
Depth Applied	.98 Ft.
Head Used	4.04 Sec. Ft.

Yield, Barley, 73 Bu. per A, -- Oats, 50 Bu. per A.

Soil, clay loam with porous stratum of gravel wash beneath.

EXPERIMENT NO. 5.

Crop, Oats -- 23.41 Acres.

Date of Irrigation	July 13-18
Duration of Irrigation	122 Hours
Area Irrigated	23.41 Acres
Water Used	35.72 Acre Ft.
Depth Applied	1.53 Ft.
Head Used	3.54 Sec. Ft.

Yield, Oats - 51 Bu. per A.

Field was irregular and difficult to irrigate. Considerable water wasted.

EXPERIMENT NO. 6.

	First Irrigation	Second Irrigation	Total
Date of Irrigation	July 6-7	July 22-24	
Duration of Irrigation, Hours	24	48	72
Area Irrigated, Acres	7.26	7.26	7.26
Water Used, Acre Ft.	2.63	7.11	9.74
Depth Applied, Ft.	.36	.98	1.34
Head Used, Sec. Ft.	1.33	1.78	

Yield - 72.75 Bu. per A.

Field much the same as described in Experiment No. 5.

These experiments show conclusively that there is a limit to the amount of water needed to mature crops. For example, in Experiment No. 6 - 72.75 bushels of Oats were grown per acre using 1.34 acre ft. of water, against 51 bushels in Experiment No. 5, using 1.53 acre ft. of water.

The proper use of a head in irrigating, is another important consideration. Much water is wasted by letting a small stream run continuously on a piece of land, instead of distributing it on the land from the sublateral as quickly as possible, and still not washing the soil. On this project, the duty of water was fixed at one and one-half acre ft. actually delivered on the land.

Closely allied to water duty, are the storage and transportation losses in the reservoir and distribution system. The two main losses in the reservoir are from seepage and evaporation. Data on seepage is very hard to get and usually

is estimated after the evaporation loss has been determined.

Evaporation varies considerably, being less in higher altitudes, and greater where there is considerable surface disturbance on the reservoir. The U. S. Geological Survey has kept a complete record on evaporation per month at different stations in the arid west. The one nearest Oakley, at about the same elevation and under similar conditions, has an evaporation of 61 inches per annum.

Data on evaporation of reservoirs in India, give values from 32 to 88 inches per annum. Mr. Strange states that the usual allowance made in Bombay projects for all losses is 48 inches, measured on the mean area of the reservoir, although in some cases it has been taken as low as 36 inches and again as high as 84 inches.

Regarding seepage, there is a great variation of opinions among Engineers. Geological and other surface conditions of a reservoir, effect seepage to a great extent, for, as silt is carried down into it, seepage undoubtedly decreases. Some Engineers have an empirical rule that seepage is one-half that of evaporation, but this does not seem logical, as there is no known relation between evaporation and the conditions that govern seepage. Other Engineers consider seepage as additional storage, and all water which seeps from the reservoir will return as the water in the latter is drawn off. This seems more reasonable than the other theory, and has been found to apply in a number of cases, among which is that of the Sweetwater reservoir in California, where upon making tests, it was found that the seepage losses were practically nothing.

In a great many reservoirs however, there is a distinct seepage loss due to the perviousness of the bottom. In various reservoirs in the west, also in India, this loss has been estimated as being from one-fourth, up to and equal that of the evaporation.

The Goose Creek reservoir has a heavy loam 30 ft. deep in its bottom and from all appearances should not have a great loss from seepage. However, as the assumed evaporation loss of 61 inches was below the average, as compared to other records in the West, the seepage loss was estimated somewhat high and assumed to equal that due to evaporation or 122 inches for both per annum.

The following table of supply and draft from the reservoir, including losses, is made from the assumed runoff in the article on Water Supply. The monthly variation in flow is based largely on observations made in 1907-08 at the time of the preliminary work.

MONTHLY SUPPLY AND DRAFT FROM RESERVOIR.

Drainage Area, 870 Sq. Miles, Assumed Run off 148.420 Acre Ft.

Month	Supplied to Reservoir AC.FT.	Evaporation and Percolation Losses AC.FT.	Drawn from Reservoir AC.FT.	Left in Reservoir AC.FT.
Oct. 1	3962	204		
Nov. 1	3829	237		3758
Dec. 1	3962	165		7350
Jan. 1	2375	170		11147
Feb. 1	21521	400		13352
Mar. 1	47570	990		34473
Apr. 1	26077	1932		81053
May 1	17440	2112	273201	105198
June 1	12275	1937	41286	93206
July 1	3962	2278	33844	62258
Aug. 1	2375	1088	27320	30098
Sept.1	3072	307	6830	4065
Totals	148,420	11,820	136,600	

In this table, Goose Creek is assumed to carry 105,000 acre ft., Trapper and Cottonwood Creeks together 30,000, leaving 13,420 acre ft. coming from Birch Creek. Considering the topography of the country, also the fact that Birch and Cottonwood Creeks require feeder canals to the reservoir, the economical design would call for two dams in Goose Creek canyon, one five miles above the other and having a combined capacity of 105,198 acre ft., according to the table.

In considering the transportation losses in canals, considerable reliable data may be had. New canals seep a great deal more than old ones, and silt which is continually carried down into the canals, will eventually stop most of the percolating losses. It would not be considered good design to allow for high losses in new canals, for the reason that during the first 2 or 3 years a system is in operation, only a small percent of the land will require water, and hence there should be a sufficient quantity in the reservoir to cover all excessive transportation losses during that period.

The Great Western Canal at Idaho Falls, Idaho, is 10 miles long, has a depth of 5 ft., a bottom width of 30 ft. and a velocity of 1.5 ft. per sec. Here, the losses for the first year averaged 3.25% per mile, but decreased gradually till the fourth year, when it was a little less than 1%. The laterals at the end of that time showed a loss of about 2% per mile. These canals were built largely in earth with some stretches in rock, the conditions not being especially favorable to low transportation losses.

The following table made from data taken on the

Phyllis Canal, in Canyon Co., Idaho, is accurate and reliable, as great care was taken to obtain it under practical operating conditions. The canal is 12 ft. wide at the bottom, about 34 miles long and built largely along a steep side hill, in coarse gravel, which naturely is conducive to heavy percolating losses.

TRANSPORTATION LOSSES - PHYLLIS CANAL

	June	July	Aug.	Sept.
Average at Head	46.2 S.F.	43.8 S.F.	43.3 S.F.	39.8 S.F.
" wasted at lower end	11.1 "	9.3 "	9.5 "	11.5 "
" diverted thru Laterals	24.6 "	30.0 "	29.2 "	22.5 "
" loss (Total)	10.5 "	4.3 "	4.8 "	5.8 "
" loss, % per mile	1.16%	.49 %	.55 %	.73 %

In Mr. Strange's work on "The Design and Construction of Small Irrigation Canals in Africa", is found the following table covering transportation to be allowed.

Canals over	100 sec. ft. capacity			.25% less per Mi.		
"	50 to 100	" " "		.50%	" " "	
"	25 to 50	" " "		1.00%	" " "	
"	10 to 25	" " "		2.00%	" " "	

Evaporation losses on canals are very small comparatively. During the irrigating season, evaporation on exposed water surface will not exceed one-third of one inch per day. On canals of large capacity, this is a negligible factor in its losses. The Main West Side Canal on this pro-

ject, for example, is designed to carry 350 sec. ft. with a water surface of 195,360 sq. ft. per mile, exposed to evaporation. The loss, due to this cause, would amount to 5425 cu. ft. per mile per day, or less than one-fiftieth of one per cent.

The transportation losses on this project were fixed at 1% for canals and 2% for laterals, all ditches with less than 25 sec. ft. capacity being considered laterals.

From a map of the distribution system the following table was constructed giving the distances from the reservoir to centers of distribution for main canals also average lengths of laterals.

Goose Creek Channel	0.5	Miles
High Line Canal, to center of distribution	5.0	"
East Side Main Canal, to center of distribution	8.0	"
East Side Laterals, average length	2.5	"
West Side Main Canal, to center of distribution	12.0	"
West Side Laterals, average length	3.0	"

Using the above assumptions as to losses in the reservoir and in transportation, the following table is constructed, giving a summary of all losses.

Reservoir as per table on page 24			11,820 A. Ft.
Goose Creek Channel	136,600 A. Ft.	@ 1 %	1,366 " "
High Line Canal	1,580 " "	@ 5 %	79 " "
East Side Main Canal	28,315 " "	@ 8 %	2,265 " "
East Side Laterals	26,050 " "	@ 5 %	1,302 " "
West Side Main Canal	104,179 " "	@ 12%	12,501 " "
West Side Laterals	91,678 " "	@ 6 %	5,550 " "
	Total		34,883 " "

From the above table it will be seen that the total losses of storage and transportation are nearly 24% of the estimated total water supply of 148,420 acre ft. This figure as compared to those of other systems, in the arid west is near the average.

Deducting the total losses of 34,883 acre ft. from the total supply of 148,420 acre ft. leaves 113,537 acre ft. of water available for field delivery, which at the fixed water duty of 1.5 acre ft. is enough to irrigate 75,700 acres of land. Notwithstanding the evident conservativeness of the above conclusions regarding water supply, duty of water, and losses, the scope of this project was limited to 50,000 acres with a provision in the State contract that a second segregation be added as soon as the water supply was proven sufficient to warrant it.

This was done for the following reasons; (1) although the computations seem conservative and reasonable, great hardship would occur if by any chance there was not enough water for all the settlers, (2) for a segregation of 50,000 acres, only one dam would be needed and in the event of an enlarged

segregation, another smaller dam could be built later at the upper dam site, (3) the cost of any future construction would be no more per acre than for the original segregation, (4) lack of available water has done more to condemn irrigation in the west than any other one thing and this then is a step in the right direction, to reestablish the confidence which irrigation deserves.

(c) THE LAND.

As stated before, the land in the Oakley valley is highly adapted to irrigation, having the physical characteristics of slope, drainage and shelter to as great a degree as perhaps any other tract in the west. The soil is an alluvial lava wash, and to some extent a disintegrated lava ash, both of which are remarkably fertile when placed under irrigation. High mountain ranges bound the valley except on the north, offering a splendid shelter, and this together with the perfect air drainage from south to north should make it an ideal fruit country. Even with the crude irrigation methods practised by the old settlers, such crops as 550 bushels of potatoes, 85 bushels of wheat, and 7 tons of hay per acre have been grown year after year. This gives an idea of what may be expected when scientific principles are introduced and an efficient irrigating system built.

Desert land around Oakley with no water rights will sell for about $25.00 per acre, while improved land with good water rights brings $100.00 to $150.00 per acre. The price of Carey Act land is virtually fixed by the State and may be different for each project, This price, which was $30.00 on the Twin Falls South Side Project, built some 10 years ago, has gradually increased until the present time, when some land has been sold for as high as $65.00 per acre. The price of the land in the Oakley project was fixed at $65.00 per acre.

Oakley is about 22 miles from the Oregon Short Line R. R., the largest carrier in the state. Through this valley

a branch could be built at a very low cost of construction, thereby furnishing an outlet, commercially, for the large agricultural crops which this valley is capable of growing.

(d) THE COST.

The question of financial backing for Carey Act projects is an all important one. The U. S. Reclamation Service built many that would never be considered under the Carey Act, because the government asks no profit for building their systems. At present financial backing is hard to get because false and reckless boosting has given irrigation a black eye. Wild cat schemes have been financed which later have failed, causing a great degree of uncertainty and doubt among promoters.

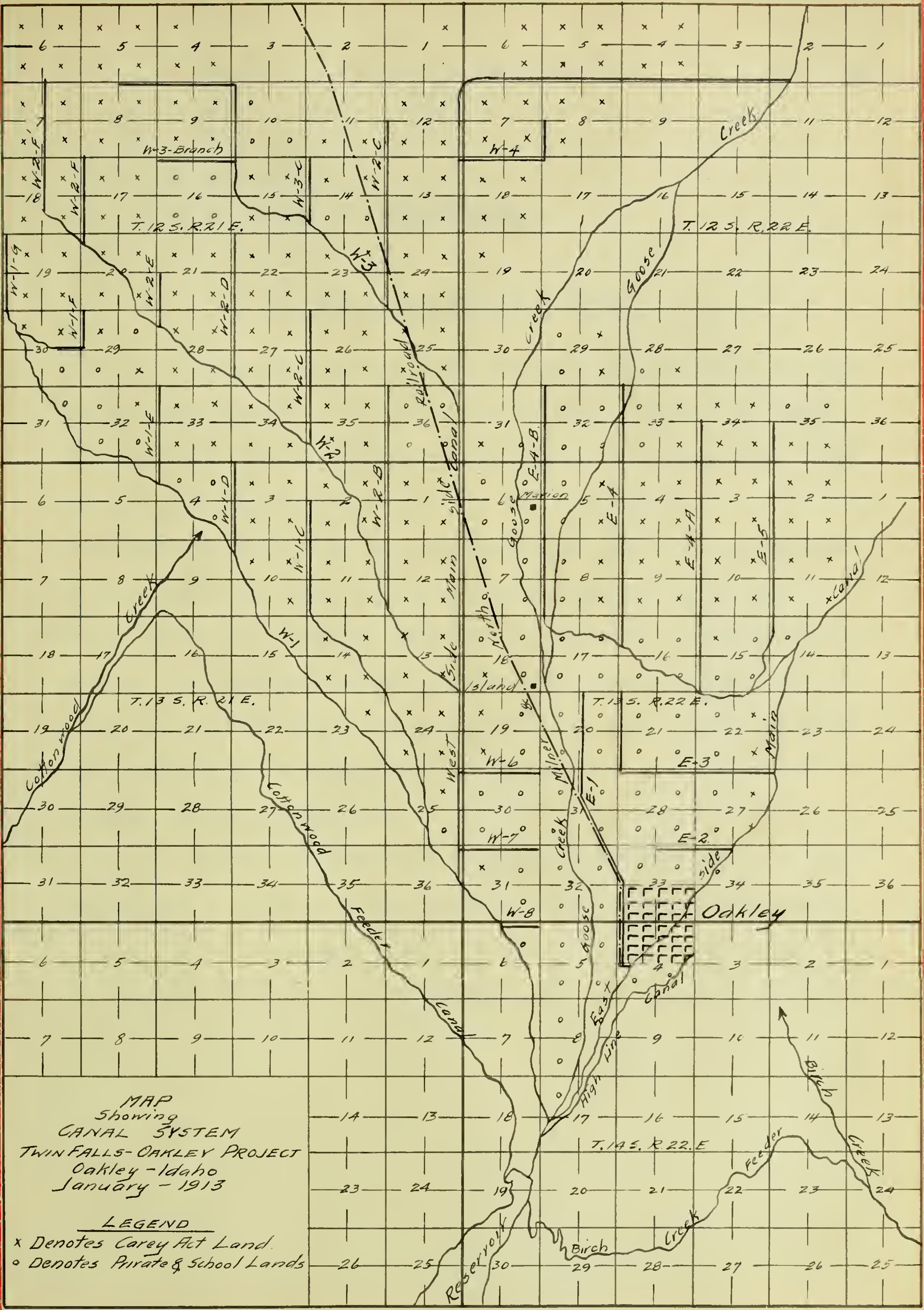
Naturally, all the water coming down Goose, Birch, Trapper and Cottonwood Creeks had already been filed on, and unless these rights were relinquished and agreements entered into, whereby all would receive water from the reservoir, the proposed project would be out of the question. The settlers consequently, realizing how the development of the community would benefit them, agreed to surrender their water rights and in lieu thereof, secure water from the reservoir at a water duty of one and one-half acre ft. This was limited to 160 acres; however anyone owning more than that amount ~~being~~ was allowed to buy additional water at the same duty for $40.00 per acre. This did not apply to Cottonwood or Birch creek water rights. In these cases, it was impossible for settlers to obtain water from the main reservoir, hence the only agreement entered into was that of getting the flood waters in and out of the irrigating season.

The following legal proceedings are necessary to

promote a Carey Act project. An individual or corporation may petition through the State to the Sec. of the Interior, to temporarily withdraw certain lands from entry for one year, in order that surveys and investigations, preliminary to filing an application for segregation, may be made. This petition must be accompanied by a check for $250.00 as a guarantee of a proposal to construct, by the applicant. If this is not done within one year from the date of the petition, the guarantee is forfeited. If the proposal is made within one year, it also must have a certified check of from $250.00 to $2500.00 as a guarantee of execution of contract. Maps, plans and other exhibits of the proposed construction must be submitted at the same time. These are investigated by the State Engr., who reports to the State Land Board regarding their feasibility. If the report is favorable and the Land Board sanctions the proposed project, formal application is made to the Sec. of the Interior for the withdrawal of the land for a period of ten years, in which time the construction must be completed.

Contracts are entered into between the State and promoter, relative to methods of construction, time limit, price at which land is to be sold, etc. The State Engineer maintains a thorough inspection of the work during the construction. Notices are sent out in advance to the incoming settlers as to when water will be available, and when proof is shown that they have developed one-eighth of their land, lived on it 30 days, besides several other requirements, such as building ditches, etc., a title is issued by the Sec. of the Interior.

It will be noted from the above, that the United States owns the land in the first place, the State acting merely as an agent for the government. Also, all Carey Act land must be desert land, which has no known mineral deposits of value.


T. 12 S. R. 21 E.
T. 12 S. R. 22 E.
T. 13 S. R. 21 E.
T. 13 S. R. 22 E.
T. 14 S. R. 22 E
W-3-Branch
W-4
W-1-G
W-1-F
W-2-F
W-2-E
W-2-D
W-3-C
W-2-C
W-1-E
W-1-D
W-1-C
W-2-B
W-3
W-2
W-1
W-6
W-7
W-8
E-4-B
E-4
E-4-A
E-5
E-3
E-2
E-1
Marion
Island
Oakley
Goose Creek
Cottonwood Creek
Birch Creek
Railroad
North side Main Canal
West side Main
Main side Canal
Milner Creek
East High Line Canal
Cottonwood Feeder Canal
Reservoir
Birch Creek Feeder

MAP
Showing
CANAL SYSTEM
TWIN FALLS-OAKLEY PROJECT
Oakley-Idaho
January-1913

LEGEND
× Denotes Carey Act Land.
∘ Denotes Private & School Lands

THE DESIGN.

1. THE DISTRIBUTION SYSTEM.

As stated before, the original segregation on this project was fixed at 50,000 acres of land. Based on these figures, the dam and distribution system were designed, and the land, being far in excess of the available water, allowed considerable choice as to its selection for the segregation. It was important that the tract be as compact as possible for a number of reasons, such as (1) cost of the distribution system, (2) transportation losses, (3) maintenance, etc. A map on page 35 shows the selection of desert land as made, the location of ditches, as built, and the private lands already under cultivation.

There are a number of methods or systems used in laying out a gravity canal distribution, and the topography of the land will invariably determine which one to employ. The Oakley system, is an example where the comparatively flat ground is made to fit the canal. In other words, the approximate location of a canal is made with reference to the lands to be watered, and a section computed that will fulfill the requirements of velocity, etc. Another method is used on more uneven land, having high ridges. Here the canals are contoured on the slopes above the lands irrigated, and by the aid of

coulees, the water may then be rapidly distributed to the farms below. These coulees usually have a hard pan near their surface that eliminates the consideration of velocity, for, even if they did wash, no harm would occur. A third method, that of using contour canals at the foot of the slopes, often misses the good lands in the pockets of these ridges.

Where the land is favorable to the first method, it is the best one to use. However, very good results have been obtained by the second method, notably at the Twin Falls-Salmon Project, where the maintenance fee last year (1912) was 45¢ per acre against 50¢ to $1.50 for other projects in this vicinity.

Before any designs can be made, however, the allowable maximum velocity in the canals must be fixed. High velocities decrease the required canal cross section and hence lower the cost of construction. But in view of the fact that future maintenance and operation constitute by far the greater factors in the cost of a project, the above statement should not be considered to any great extent in design. The maximum velocity will depend chiefly on the character of the soil and should not be high enough to scour and destroy the canal section. On the other hand, it should not be so low as to allow the deposition of silt and the growth of weeds, which interfere with the canal flow. Contour ditches must have no sharp curves, for these retard the velocity and cause eddies that wash the banks, etc. The beginner, locating this type of canal, will invariably follow the contour too closely, instead of approximating it. This degree of approximation, however, is an

important one, especially on rough land, and can only be mastered by experience.

Velocities in canals vary largely with the depth of water in them, Large ditches in earth, carrying 1000 sec. ft. or more, require a slight fall of from 8 to 16 inches per mile and very seldom have a velocity as high as 3 ft. per sec. Smaller canals and laterals may often have velocities of 3 and even 3.5 ft. but this should be governed entirely by the nature of the material through which they are built.

The soil in the Oakley valley is quite compact, rather fine, but heavy, and should be able to withstand a comparatively high velocity. The Twin Falls-Salmon Project, Idaho, has a maximum velocity of 2.75 ft. per sec. for their system; the Twin Falls North Side project, Idaho, 3 ft. per sec.; and the Minidoka Project, Idaho, 2.5 ft. per sec. All of these tracts have about the same kind of soil, being located within a 75 mile radius of the Oakley project. A maximum velocity of 3 ft. per sec. was then decided on for the local System.

After some years of operation, the velocity is ~~e~~affected to a considerable extent by the foreign growths and deposits in the canals, such as weeds, slimy moss, silt, sand, etc. In some experiments, referred to later, it was found that several canals designed for a 3 ft. velocity, were capable of even higher velocities, due to this foreign growth and deposit. Here at Oakley, after a short period of operation, it was found that 2.5 or possibly 2.75 ft. per sec. should have been the maximum velocity, for considerable scouring

occurred, especially on the smaller canals. In the matter of design it is better to provide for the least favorable condition of the canals, namely when they are new, than to rely on a more suitable state, such as mentioned above, after they have been in operation for some time. Under ordinary conditions, a velocity of 2.5 ft. per sec. will prevent the growth of weeds, also the deposition of silt to any great extent.

In this connection, it is important to note that canals may be designed under any maximum velocity, mathematically, by computing a section which has the required amount of wetted perimeter to reduce its velocity sufficiently. Such abnormal sections are very impracticable. No ditch, for example, with an 8.0 base and 0.4 ft. depth of water will ever operate as such, but instead, form narrow channels in places that have an increased velocity which destroys the canal section. The ideal section, as regards seepage, would be one having a minimum wetted perimeter, but such a design is impracticable and a trapezoidal section is used largely in earth and loose rock. It is obvious that some rule regarding depth of water, as compared to the canal base, should be made. On the Oakley System this ratio was not fixed at the time of the original design, and was the cause of considerable expense later on, when it was found necessary to rebuild all abnormal sections.

The slopes on all earth ditches, with very few exceptions, were made 3 to 1 and 2 to 1, respectively, on the inside and outside of banks, and were found to be satisfactory for this kind of soil. For an economical section, as regards

cost, the excavation was made equal to the embankment as much as possible. This usually brought the water surface up on the fill, requiring wide crowns of 6 ft. on the main canals and 4 ft. on the laterals. Free-board above the water surface was made 1.5 ft. on main canals and 1.0 ft. on laterals. On contour laterals, no upper banks were built, the water section being wholly in excavation. This was done to allow surface water to drain in from above, without cutting the banks.

After deciding on the maximum allowable velocity, the next question was that of a formula to compute the capacity, etc. of canals. In the United States, Kutter's well known formula $V = C\sqrt{R\ S}$, is used almost exclusively by irrigation Engineers. The determination of the right value of "N" to use in this formula is often quite difficult, unless experiments are resorted to. This coefficient of roughness for earth canals in good condition, is usually assumed as .025, but in reality varies with the soil, the amount of gravel or rock in it, the uniformity of its cross-section, the time the canal has been in operation and the growth and extent of foreign material on its surface.

Experiments, conducted on the canals of the Twin Falls North Side project, Idaho, brought out surprising results in this regard. Earth canals, which had been designed with "N" = .025 were found to be operating with a coefficient from .017 to .021. The banks and bottoms of these canals had a vegetable growth of weeds and moss, that naturely led one to expect a coefficient of more than .025 instead of less. But, upon close examination, it was found that the weeds were

above the water surface, the fungus growth was very slick, and that the silt had made a very uniform cross section, all of which explained the existing low coefficient. In obtaining the required data, the water was carefully measured at each end of the experimental section; these stations being so chosen that an accurate measurement could be taken of the canal-sections. Current meters were used to measure the amount of flow, and levels were taken for a water-surface profile, checking it against the canal gradient. With this data the elements of the canal were computed, namely, velocity, area of section, wetted perimeter, hydraulic radius, and quantity of water flowing. The next step, then, was the computation for "C" in Kutter's formula. After this was found, and by using the previously observed slope of the canal water-surface, the coefficient "N" was computed. As a common rule, .025 is the value for the coefficient of roughness used in the design of most new earth canals and was the one adopted for the Oakley System.

Later, the Consulting Engineer, being of the opinion that .025 might be too high for certain canals on the tract, caused several experiments to be made in the vicinity of Oakley to determine the coefficient "N" on old canals still in use. The same methods were followed in these experiments as in those of the Twin Falls North Side project, mentioned above, with one exception. These canals were in poor condition, generally, and had no regular cross section, making it hard to get accurate measurements. The wetted perimeter was determined by assuming the section a rectangle; the width plus twice the

average depth approximating the actual perimeter.

Experiment No. 1, gave a coefficient of "N" = .028. The alignment of this section was fairly regular, the sides and bottom being free from weeds, but strewn with stones, 0.5 to 1.5 inches in diameter. The depth and width of the section was regular.

Data. --- V = 3.00, S = .024, R = .43, C = 34.8
Base = 4.5, Depth = .36, N = .028

Experiment No. 2, Here the bottom of the canal was silted over considerably, and conditions generally were about the average to be expected on ordinary canals.

Data. --- V = 1.29, S = .0022, R = .43, C = 42.6
Base = 10.0, Depth = 0.5, N = .026

Experiment No. 3, is perhaps more reliable than the other two, because of the better conditions existing here for accurate observation. The alignment was straight, the sides and bottom were regular, and small stones laid evenly on the bottom. There was no vegetable growth.

Data. --- V = 1.02, S = .002, R = .32, C = 40.3
Base = 11.0, Depth = .34, N = .0247

Judging from the results obtained in the above experiments, the value "N" = .025, as fixed for the Oakley System is quite near the actual figures for this coefficient. It will be noted that all these canals which were experimented on, had the socalled abnormal sections. These occurred only in certain places, and were chosen for experiment because of

their large wetted perimeter.

The transportation losses used in design were the same as those stated in the previous article on Duty of Water, being 1% per mile for all canals carrying more than 25 sec. ft. of water, and 2% per mile for those carrying less than 25 sec. ft. The Twin Falls North Side Co., also made experiments on small earth canals for losses, and obtained results, while varying considerably, due to the nature and condition of the canals, still averaged quite closely those assumed for this project. In these experiments also, the water was carefully measured with current meters at each end of the experimental section.

Considerable preliminary field work must be done before a distribution system can be laid out properly. First, all section lines must be run out and located, and then all section corners and quarter corners permanently re-established. This will make the final location of ditches much more accurate, also easier when the preliminary is finished. Bench marks should be placed on all corners and quarter corners, that checks may be had on the surveys at least every half mile. A topographic map of the proposed segregation should then be made showing contour intervals 5 or 10 ft. apart, depending on the unevenness of the land. On the Oakley tract, with its exceptionally flat slope, a 3 ft. interval would have been even better. It could easily have been made from profiles taken of the north and south section and quarter section lines.

On such a map, the design may be approximated quite closely by a paper location, at least, a good idea will be

obtained of the lay of the land, which will make serious errors in location impossible. For some unexplained reason, no such map was made here, and it was not long before its need was felt. Section line laterals were, in many places, located on very steep grades, which made a resort to the so-called abnormal sections necessary. The State, however, condemned them, compelling the use of concrete drops and checks to reduce the grades. The canal sections were re-proportioned and a ratio of water depth to canal base agreed upon to be 20% or more. Unquestionably, 25% would have been a much better minimum than 20%.

Section line ditches have these advantages; they minimize the length of ditches in the system and cut up the segregation most satisfactorily. Practically, they are only possible under nearly ideal land conditions, which very seldom occur. More than 12 ft. fall in a 1000, makes this type of a ditch almost out of the question, if it is to carry the ordinary amount of water which laterals do, and have a proper cross section. Furthermore, because of the steep slopes, these ditches are often too deep in the ground, making it hard for a farmer to get his water, unless he takes it out some distance higher up on the canal, and this means unnecessary extra ditches.

Finally, the design and computations on all canals should be made in the office, from the preliminary notes and maps, and not in the field by the locating party, unless in some instances this becomes absolutely necessary. Too many factors enter into the efficient design of canals and these

should not be decided upon at the spur of the moment, in the field. Serious mistakes may creep into such work, that is not carefully checked, such as wrongly computed sections, or wrongly assumed grades, which are expensive errors to correct, once the ditch is built.

There is no doubt but what, with a good topographic map, an entirely different design of a system would have been made here, than the one which was built. The contour system should have been used more, expecially on the steep land between W_1 and W_3, with perhaps a few stub ditches on the section lines that would not have had any considerable capacity. Below W_3, the land is much flatter and section line laterals might properly be used. On the east side, contour laterals should have been used entirely, except, where it was necessary to follow the old ditch system, on account of the cost of right of way incurred if changes were made.

The Carey Act law requires that the canal system shall so distribute the water that no settler will be required to build more than 0.5 mile of ditch from the nearest lateral to the high point of his land.

A map is then necessary showing the size and location of each farm, as the public sale of Carey Act lands, usually takes place before the work of construction has begun. In the case of the Oakley project, practically all available land was sold at that time. If the lands are not all sold, the farm unit is assumed as 40 acres for the unsold sections.

The usual distribution of water for irrigation is that of rotation, each settler receiving his water at certain

periods, previously arranged. To meet the demands of crops, the amount of water necessarily varies for different months. On the Oakley project this was fixed as 20% in May, 30% in June, 25% in July, and 20% in August, leaving 5% for early and late irrigation, domestic use, etc. See table on page 24. Assuming an irrigation period of 24 days per month, a section of land (640 acres) at a water duty of 1.5 acre ft., will require 288 acre ft. as 30% of its allowable water. One second foot of water, running continuously for 24 hours, will amount to very nearly 2 acre ft. and consequently, 6 sec. ft. will then be required to supply 288 acre ft in 24 days. Assuming a 24 day run, allows several extra days to supply any unexpected demands that might occur.

Knowing the flow in sec. ft. required to irrigate one acre of land, also the transportation losses to be considered, canals can then be designed for capacity by referring to the ownership plat mentioned above. See page 35 for a map of the Oakley tract.

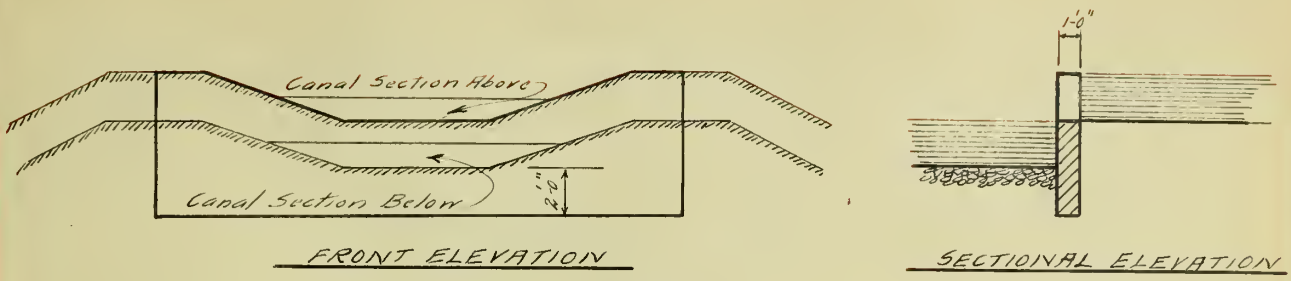
The location of the West Side Main Canal down through the center of the valley, made construction quite expensive, 34 concrete drops being needed to keep under the maximum allowable velocity of 3 ft. per sec. This is probably the best location, as it brings water quickly to the contour laterals W-1, W-2, and W-3, which in turn feed the section line laterals. This canal will water all lands in the segregation west of Goose Creek or approximately 32,000 acres.

To water the rich land to the northeast, the East Side Main Canal was built, as shown on the map, page 35. It

irrigates approximately 17,000 acres of land, including most of Oakley. The location of this canal was governed largely by the old canal, which served this territory; and was accomplished by using checks, which dropped the canal into the old channel just before entering Oakley, thereby avoiding the purchase of expensive right of way through the town. The laterals of the East Side Main Canal have largely a section line location. A High Line Canal was found necessary to water a small part of Oakley and some high land above the East Side Main Canal, or approximately 1000 acres in all.

As stated before, considerable trouble was experienced with section line canals, and great expense was incurred in building drops and checks to lower the original grades, which, in some instances, were 20 ft. to the 1000, also in changing the canal section that it come within the 20% specification, demanded by the State. In fact, the expense was so great that it is safe to say that intermediate contour laterals could have been built, taking the place of section line laterals, which would have given better service at a less initial cost, as well as future operation. All this, emphasizes the need of a topographic map, the use of which would very largely have eliminated such errors as these.

The table on page 54 includes all canals in the present system, showing their capacity, canal base, depth of water, and grade. These elements are taken just below the headgate of each canal. Again referring to the map on page 35 attention is called to the fact that the laterals have dead ends. Such a condition should not exist, for under the

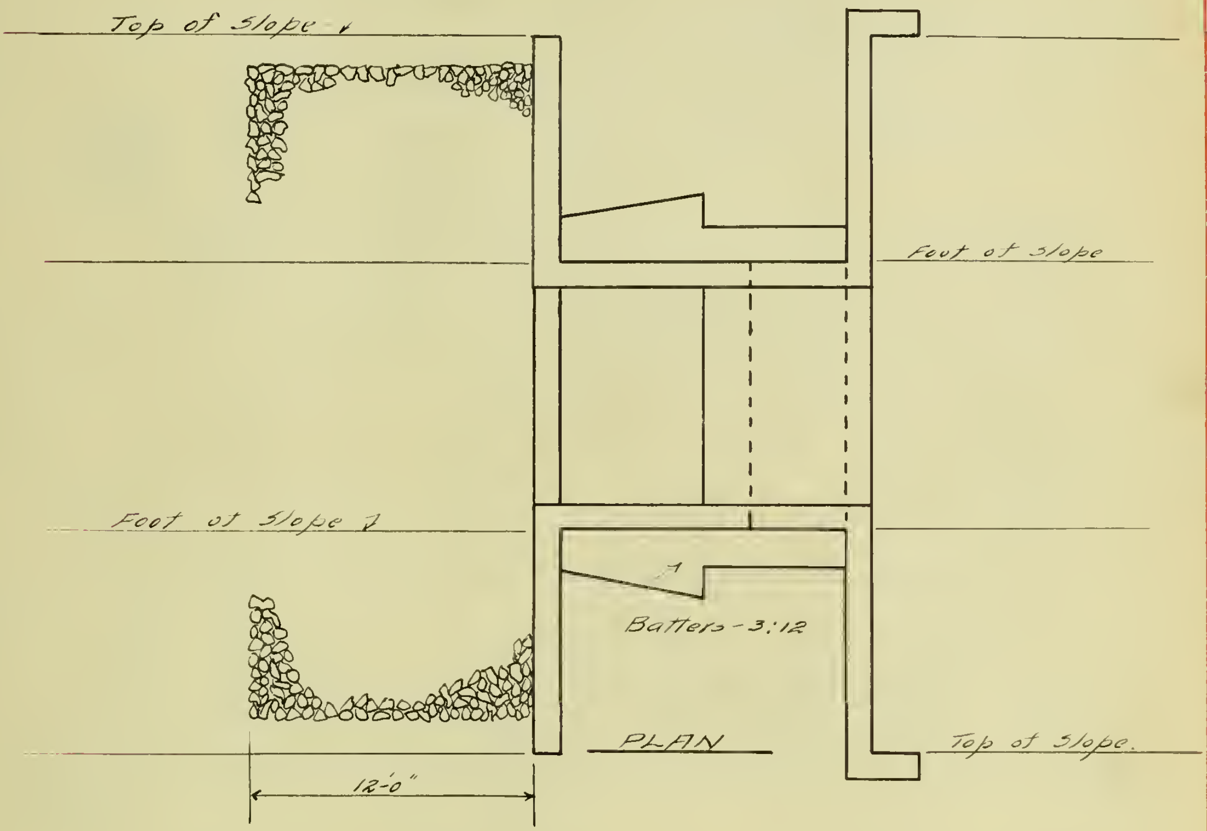


Canal Section Above
Canal Section Below
2'-0"
1'-0"

FRONT ELEVATION

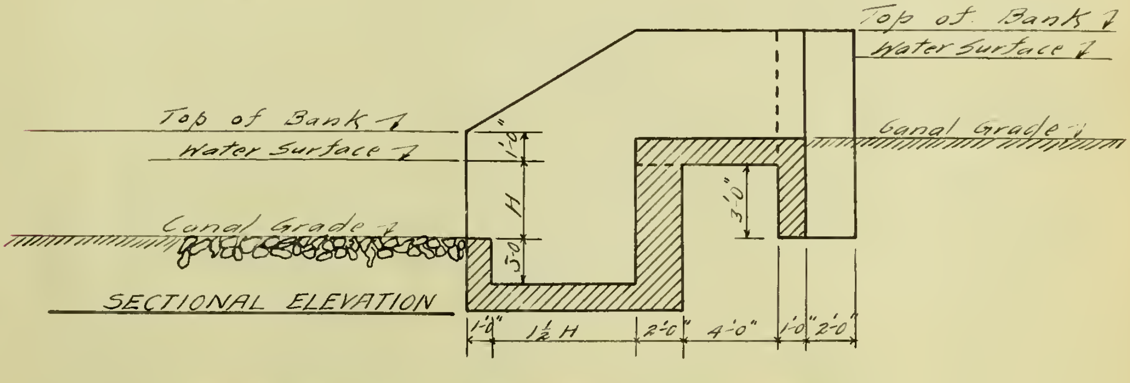
SECTIONAL ELEVATION

CONCRETE CHECKS



Top of Slope
Foot of Slope
Foot of Slope
Batters - 3:12
Top of Slope.
12'-0"

PLAN



Top of Bank
Water Surface
Canal Grade
Top of Bank
Water Surface
Canal Grade
1'-0"
H
5'-0"
3'-0"
1'-0"
1½ H
2'-0"
4'-0"
1'-0"
2'-0"

SECTIONAL ELEVATION

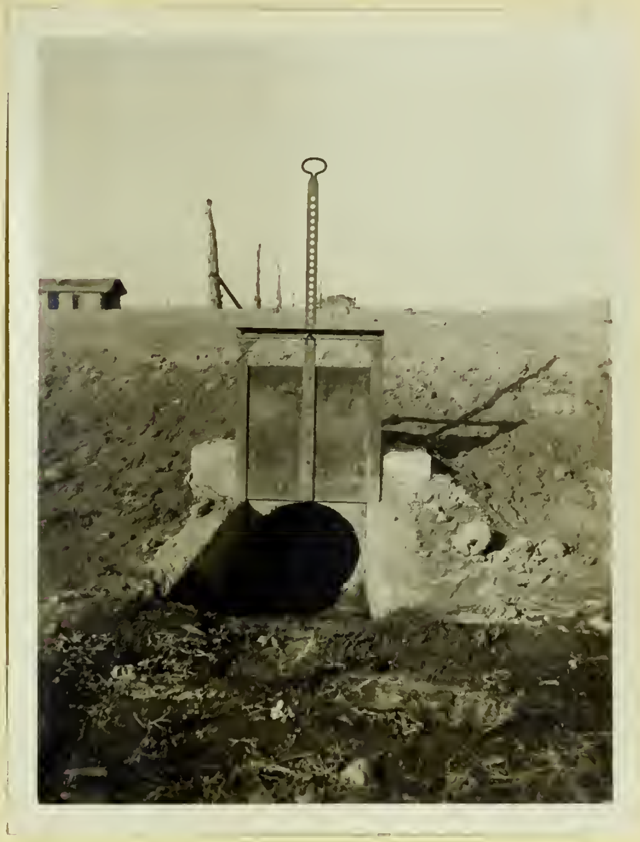
CONCRETE DROP W. S. MAIN CANAL

principle of rotation of water distribution, this will cause serious trouble, if no outlets or drains to the canal below are made.

In ditches carrying less than 5 sec. ft. checks were constructed by heavily rip rapping a steep section between two others that were reduced to satisfactory gradients. On the larger ditches concrete drops were built. Two designs were used and are shown by drawings on page 48, the upper one being that of drops used on the West Side Main Canal, and the lower one, of drops used on section line and other laterals, to make possible their original location.

The drops in the West Side Main Canal should have been designed with a sump 2 ft. deep instead of 3 ft. These sumps will invariably fill with a silt that prevents the impact of falling water against their bottom. The batters on the side walls of these drops are 3 to 12. The structures are built largely in excavation, which, should not require such a heavy batter as used, giving practically a gravity section. The size or width of all drops was computed from Francis' formula, $Q = 3.33 \, l \, H^{\frac{3}{2}}$ for weirs with no end contractions.

On the other type of drop, the cut-off wall should have been carried 3 ft. below the bottom of the lower ditch instead of 2 ft., to insure greater stability for the structure, also have carried the ends of this wall back 3 ft. at right angles, up into the banks of the upper canal to provide against water following the concrete face to the outside of the bank, causing a washout. As built, these drops were merely a vertical



LATERAL MEASURING GATE.

CONCRETE INCLINE.

wall, 1 ft. thick, set transversely in the canal and having a notched section, similar to that of the canal above. A heavy rip rap, just below the drop and taking the place of a sump, receives all the impact of falling water.

On page 50 is shown an illustration of the measuring gates to be used on this project. The frames are built of steel and set in concrete bulkheads, a steel corrugated pipe carrying the water under the bank. Two sizes of pipes are used, vis., 12" and 18" diameter, ~~and~~ In the case of the large laterals, the headgates are composed of a battery of the 18" pipes. Each farmer will have his own measuring gate under the direct control of the water master for the district.

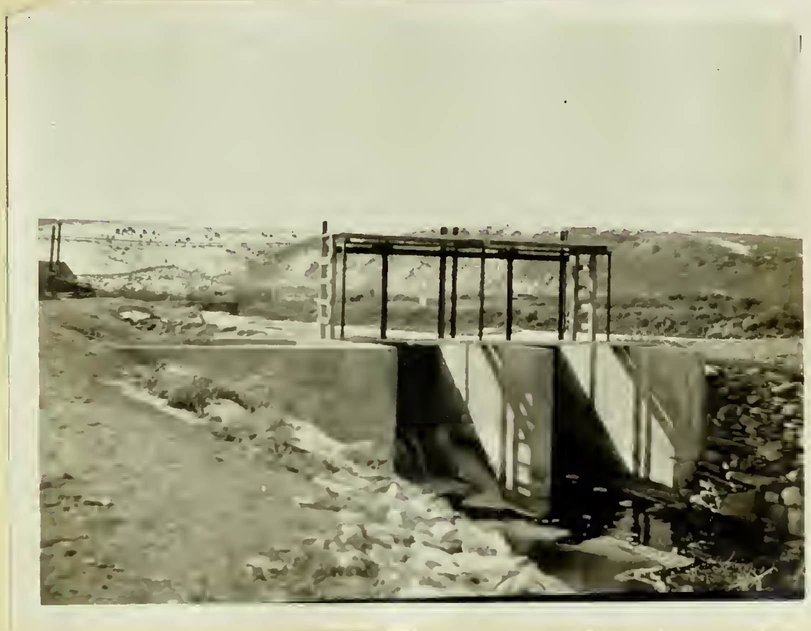
From experiments it is proposed to make a chart or table showing the discharge for various heads of the canals and openings of the gate. The operation of this theory will always prove unsatisfactory, in that the head will vary considerably in the time of an average irrigation run and no great reliance can be placed on such water measurements. Further, the grade of the pipe outlet is the same as that of the lateral and trouble will be experienced in the form of a back head. The use of corrugated pipe should also be condemned. There are no excessive pressures, either internal or external, to consider; and on the other hand the discharge coefficient for these pipes is very low. Several tests have been made on an 18" pipe, which gave from 0.55 to 0.58 for this value. Weirs are, in the opinion of most Engineers, a sufficiently accurate, and surely the least expensive, means of measuring water under ordinary conditions. When water becomes

scarce, however, a corps of hydrographers, using current meters may become necessary, as in the case of the project in the Imperial valley, California. The Cippoletti Weir is used on all U. S. Reclamation projects to measure water for the farmers.

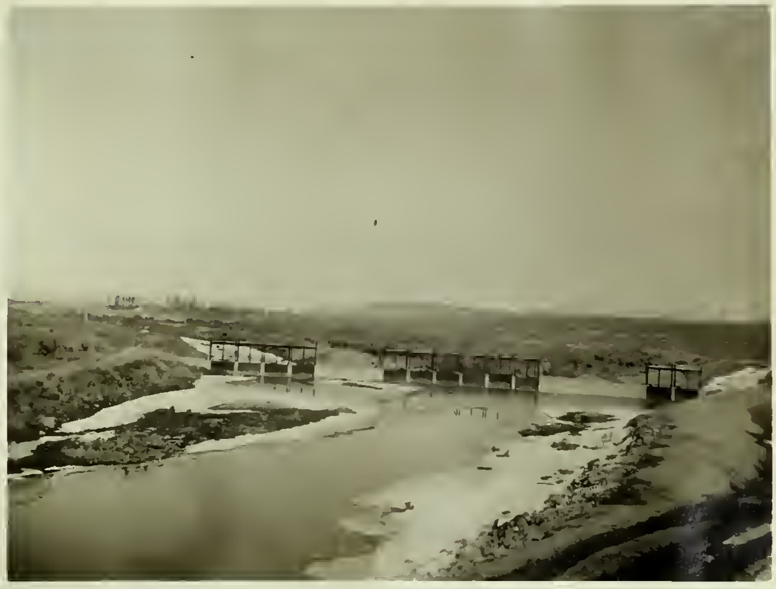
The wagon and railroad bridges used on the project are substantial structures, the former having a clear width of roadway of 16 ft. and are designed to carry a 15-ton traction engine. The railroad bridges across the canals are of a wooden stringer design, following closely the standard practice of eastern roads.

The main head gates and diversion weirs are located about 1/4 mile below the dam, the structure being made of concrete with steel gate frames. The steel gates are operated by hand power, using a ratchet. The hoisting devices are racks and pinions, the former being fastened to the stem of each gate. The West Side and East Side Main Canals take out at these head-gates, and any waste or overflow water, coming from the spillway, is diverted here into Goose Creek Channel. Two illustrations of this structure are shown on page 53. The High Line Canal takes out above the Flour Mill, thereby gaining the 14 ft. in elevation. See map, page 89.

For a summary of the unit contract costs for the Distribution System, see page 68.



E. S. MAIN CANAL HEAD GATES.
(Looking Upstream)



MAIN HEAD GATES DISTRIBUTION SYSTEM.

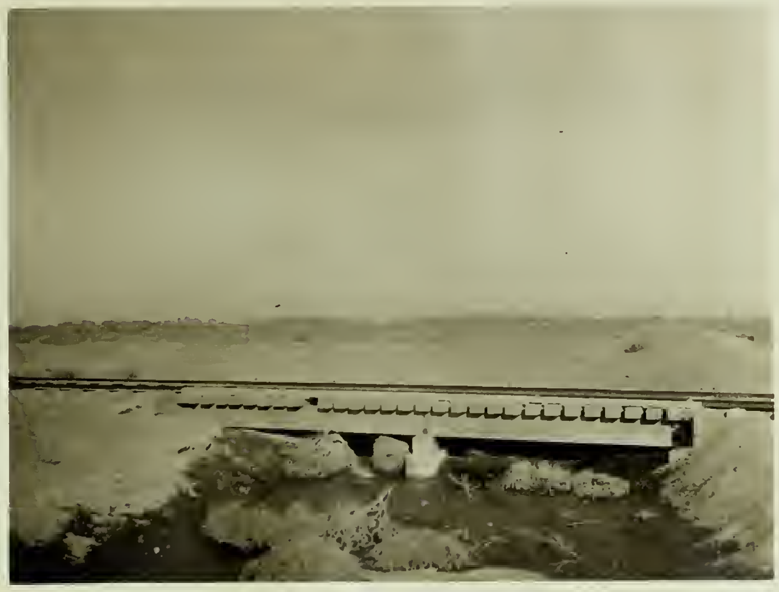
TABLE OF ELEMENTS.

OAKLEY CANAL SYSTEM.

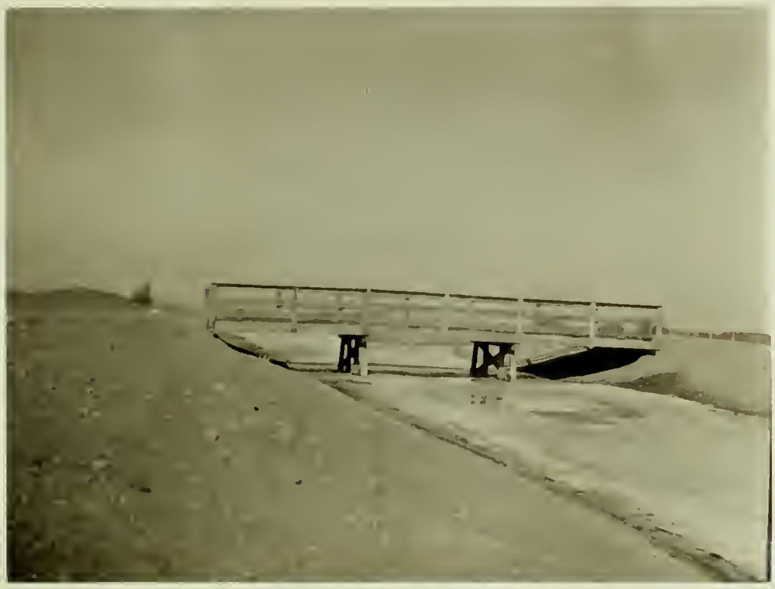
Canal	Capacity Sec.Ft.	Base. Ft.	Depth of Water-Ft.	Slope per Unit Length
W.S.M.C.	370.0	24.0	4.0	.00045
W-1	89.10	5.0	2.7	.0010
W-1-C	27.03	5.0	1.2	.0030
W-1-D	13.5	5.0	0.7	.00666
W-1-E	11.14	5.0	0.57	.0104
W-1-F	5.61	3.0	0.50	.0100
W-2	110.0	10.0	2.5	.0100
W-2-B	22.40	5.0	1.0	.0045
W-2-C	12.25	5.0	1.0	.0020
W-2-D	15.96	5.5	0.7	.0076
W-2-E	6.60	3.5	0.6	.0053
W-2-F	6.5	4.0	0.5	.0082
W-3	68.40	9.0	2.0	.0010
W-3-B	17.20	5.0	1.0	.0030
W-3-C	10.21	4.0	0.9	.0020
W-3-Br	6.42	4.0	0.9	.0008
W-4	7.44	5.0	1.0	.0005
W-6	6.00	4.0	0.5	.0070
W-7	9.00	4.0	0.6	.0076
W-8	4.76	4.0	0.34	.0200
E.S.M.C.	186.48	12.0	3.0	.0010
E-1	18.09	4.0	0.9	.0060

TABLE OF ELEMENTS. (CONTINUED)

Canal	Capacity Sec. Ft.	Base Ft.	Depth of Water-Ft.	Slope per Unit Length.
E-2	6.5	4.0	0.5	.0082
E-3	18.09	4.0	0.9	.0060
E-4	101.25	6.0	2.5	.0015
E-4-A	24.0	5.0	1.1	.0035
E-4-B	25.5	6.0	1.7	.0050
E-5	20.7	4.0	1.2	.0025



RAILROAD BRIDGE.



WAGON BRIDGE.

2. FEEDER CANALS.

The feeder canals, Cottonwood and Birch, were designed to carry flood water from these two creeks to the reservoir. In either case, this maximum was assumed as 100 sec. ft. The heavy spring run off comes rather late at Cottonwood as compared with Little Cottonwood Creek, some 5 miles nearer the reservoir, because the former has a comparatively long drainage basin, while that of the latter is short and steep. Having their run offs at different periods, allows one 100 sec. ft. capacity canal to serve both of these streams.

Both feeder canals are of the contour type, as they follow the foothills on either side of the valley. Concrete inclines were used on the Birch Creek Canal to allow a greater choice of location, while on the Cottonwood Feeder, which is somewhat larger, more difficult construction was encountered. Nearly 11,000 ft. of steel flume on a wooden substructure, as high as 96 ft. in places, were required, also a heavy, thorough cut, 30 ft. deep and 400 ft. long, besides 1400 ft. of tunnels 4x6 ft. in sections.

The canals proper, outside of some sections in solid rock, were for the greater part in hard earth that had considerable rock in it, and hard-pan near the surface. Slopes of 2:1 and 1:1 respectively were used on the inside and outside of the lower bank. The water section being entirely in excavation, required no upper bank, also allowed the many draws and coulees to draw into the canal without cutting its bank. To provide for an entrance head into the tunnels a 2.0 ft.

drop was made in the grade at each upper portal.

Considering the nature of the canals, the pervious earth material, the tunnels, as well as the rock sections, with their fissures and crevices, one is naturally confronted with the question of the feasibility of a concrete lining. From an economic standpoint, there is no doubt but what such construction should not be attempted, for the present at least. These canals have cost a considerable sum already, perhaps too much, and before further expense is incurred, the available water should be more accurately ascertained. Only a valuable crop would warrant the extra expense here.

The U. S. Reclamation Service in California has made several experiments on the relative efficiency of earth and concrete lined sections with the result that the latter is approximately 8 times as efficient as the earth lined canal. Under ordinary conditions a concrete lining for ditches is not to be encouraged, except when the scarcity of water makes it very valuable. Such a lining will cost approximately 75¢ per sq. yd. in place.

The structures needed on these feeder canals were diversion dams, flume trestles and bulk heads, also concrete drops. The diversion dam, a drawing of which is shown on page 77, is built of concrete, has a spillway sufficient to discharge any flood water above the capacity of the feeder canal, also a flash-board-gate control of the amount of water to be diverted into the feeder. The structure is built on a firm gravel and boulder foundation, and is designed to withstand a water head that spills 1.5 ft. deep over the weir.

On page 81 are two illustrations showing the method of construction used on a high flume trestle. It also shows the standard type of structure used, this being a tower design. It will be noted that there are intermediate girts or running braces between towers. These were used only in erection. In general, the stringers and knee braces are 4"x8", sway bracing 2"x8", girts 3"x6", posts 6"x6" and the flume carriers 4"x4". Bent posts have a batter of 3 in 24 and bents are on 23'-6" centers, with the posts setting on concrete pedestals. This design, while sufficient to withstand all the computed stresses, is nevertheless, weak for the following reasons. First, the batter is not heavy enough for this windy country, to be considered very safe; second, the span of 23'-6" between bents is too great for rigidity in the stringers, but will cause them to bow out when under load; third, at least the two lower decks on the 96' ft. high structure should have had 8"x8" posts; and fourth, 2"x6" sway bracing, properly nailed, would have served equally as well as the 2"x8" and offered 25% less wind resistance.

To provide for an entrance head to increase the velocity in the flumes to 7 ft. per sec., from a maximum of 3 ft. per sec. in the canals, the grades of the flumes were lowered 0.6 ft. and a concrete intake designed, 20 ft. long, fitting the ditch section at one end and the flume at the other. In the same way, to reduce the velocity upon leaving the flume, concrete outlets having the general shape of the inlets, but 40 ft. long, were designed. The theory is that in this 40 ft. the water will loose its high velocity in raising 0.6 ft. to

the ditch section.

The drops used on the Birch Creek feeder canal are made of concrete, being merely inclined chutes that carry the water down a side hill, and discharge it into a basin formed by throwing a dyke across a coulee. At this point the canal begins again on its normal gradient. An illustration of this type of drop is shown on page 50.

The spillway at Little Cottonwood is a concrete structure which provides for the waste of any flood water which the feeder is not capable of carrying. The gate regulation here is of the flash board type, similar to that used on the diversion dam at Cottonwood Creek.

On page 68 is given a summary of unit contract prices for these feeder canals.

3. RESERVOIR AND DAM.

During the preliminary of '07-'08, investigations were made for a suitable reservoir site. Two were found in Goose Creek Canyon, about 5 miles apart, the lower one being nearly 4 miles above Oakley. Surveys were then made, from which topographic maps were drawn, using a 10 ft. contour interval. With a planimeter, the areas at the different contours were found, and the capacities of the reservoirs computed. The computations showed the upper site to be preferrable, in that a dam built here would cost less per acre ft. of its capacity, but the site had to be abandoned because of the inaccessible depth of the bed rock in the creek bottom. At the lower site, bed rock was found at a maximum depth of 27 ft., and a dam was then designed for this location.

The table of Reservoir Supply and Draft, on page 24 includes a segregation of 75,700 acres of land. Using a direct proportion for a 50,000 acre segregation, gives 70,000 acre ft. necessary to be stored. The size or height of the dam, then, must be such that the reservoir capacity, the figures for which are obtained from the topographic map described above, will equal that of the maximum amount of water to be stored in it at one time, or 70,000 acre ft.

From test pits, it was found that the geological conditions were nearly ideal in the reservoir, as the underlying strata of rock took the shape of a dip or saddle, thereby making seepage losses between strata almost impossible. Conditions exactly opposite to these occur in the Jerome

Reservoir, Idaho, where the strata lies in an inverted saddle and after a certain height of water is reached during storage, the bottom seems to drop out. The greater part of the water escapes between the beds of rock and comes to the surface more than a mile distant.

The following table gives the storage increment for each contour interval, also totals to each contour.

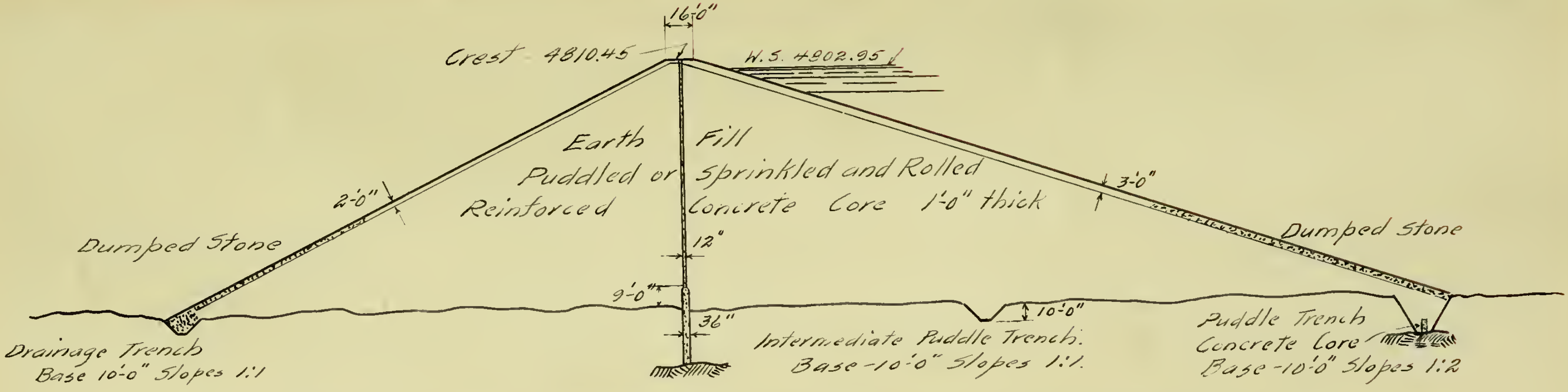
RESERVOIR CAPACITY.

Elevation Sea Level	Depth Ft.	Increment Acre Ft.	Totals Acre Ft.
4666.95	0	0	0
72.45	5.5	90	90
82.45	15.5	681	771
92.45	25.5	1621	2392
4702.45	35.5	2559	4951
12.45	45.5	3257	8208
22.45	55.5	3922	12130
32.45	65.5	4747	16877
42.45	75.5	5523	22400
52.45	85.5	6395	28795
62.45	95.5	7366	36161
72.45	105.5	8190	44351
82.45	115.5	8790	53141
92.45	125.5	9592	62733
97.45	130.5	5180	67918
4802.95	136.0	6434	74351

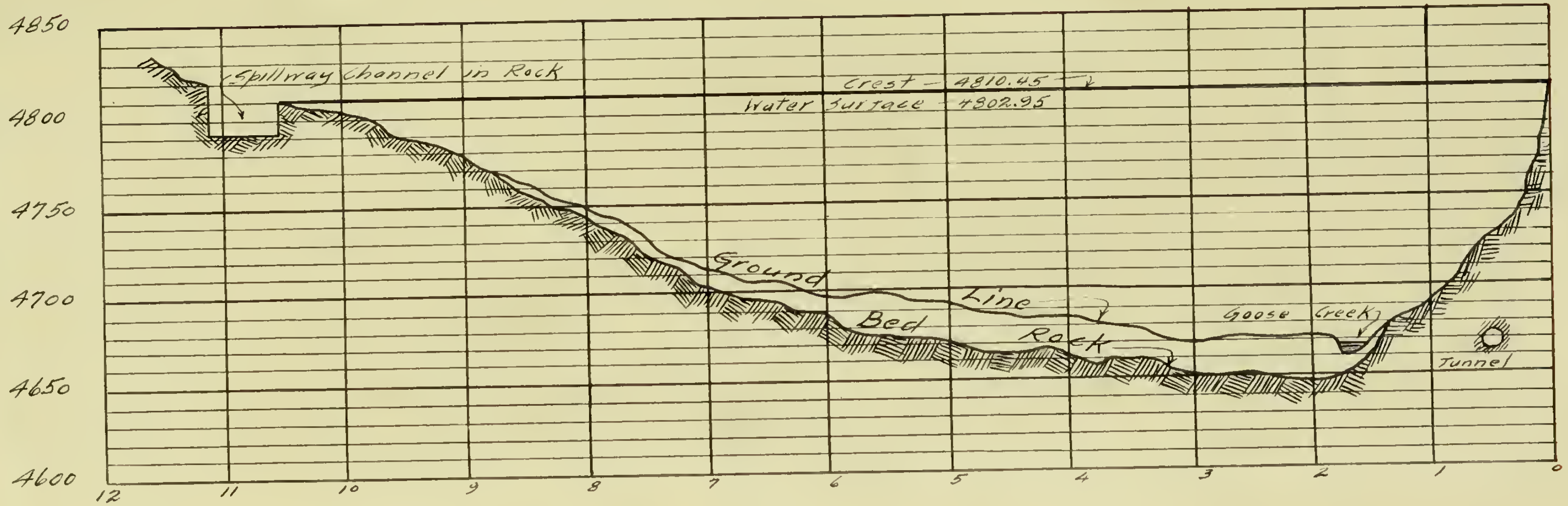
The three most important conditions which govern the type of dam to be built are (1) the foundation available at the damsite, (2) the height and length of the proposed dam, and (3) the cost of building materials. At the Oakley damsite, the width of the canyon would have made the cost of a masonry dam very expensive, because of the great yardage required in it. In the first place, suitable stone was not available in the vicinity, and also the cost of cement, shipped in from Utah, discouraged this type of dam. The nature of the soil deposits at this damsite, also the favorable distance of bed rock from the ground surface, led to the adoption of an earth dam design, with a reinforced concrete core, the concrete base of which rests on bed rock.

Earth dams are as a rule more stable than any other type, if built properly. With their flat slopes and wide base, failure through sliding or overturning are practically impossible. Erosion,/ by escaping water, due to an inadequate spillway, or the improper construction of the embankment, is the only way in which the structure is liable to fail. The Oakley dam, having the distinction of being the highest earth embankment ever built, led to many special precautions, which are more consistently explained in the next chapter, on Construction.

It has been stated in connection with this dam, that the sole function of the core wall is to prevent animals from burrowing through the structure. On the contrary, there is no question but what the chief purpose of any type of core is to prevent, or at least resist seepage, and even a 1 ft. wall


16'-0"
Crest 4810.45
W.S. 4802.95
Earth Fill
Puddled or Sprinkled and Rolled
Reinforced Concrete Core 1'-0" thick
2'-0"
3'-0"
Dumped Stone
Dumped Stone
12"
9'-0"
36"
10'-0"
Drainage Trench
Base 10'-0" Slopes 1:1
Intermediate Puddle Trench:
Base-10'-0" Slopes 1:1.
Puddle Trench
Concrete Core
Base-10'-0" Slopes 1:2

MAXIMUM SECTION OF DAM ON EARTH FOUNDATION


4850
4800
4750
4700
4650
4600
Spillway Channel in Rock
Crest - 4810.45
Water Surface - 4802.95
Ground Line
Bed Rock
Goose Creek
Tunnel
12
11
10
9
8
7
6
5
4
3
2
1
0

PROFILE ON ℄ OF DAM

will act in this manner to a certain extent. The theory that water from the reservoir will not penetrate the fill and reach the core wall is not substantiated by the results of experiments on earth dams elsewhere. On the earthen dams in the Croton Valley, which provide storage for the New York water supply, it was found that water from the reservoir, not only reached the core but passed on through it as well. To cope with this condition, it would then be well to make the side of the dam, up-stream from the core, as impervious as possible by the proper choice of materials. On the down-stream side, a much coarser material should be used, along with proper drainage trenches in the bottom of the structure, thereby, providing a rapid drainage for any water that passes through the core wall. The level of the ground water will thus be lowered, which, at the same time increases the stability of the structure, by reducing the upward pressure of the seepage water.

That an earth dam seeps, does not necessarily condemn it as an unsafe structure, as in that event, nearly all earth dams would be dangerous. In the Oakley dam, seepage was detected after the water stored in the reservoir reached a head of 40 ft. This water, some 10 inches or one-fifth of a sec. ft., comes out of the drainage trench at the lower toe, perfectly clear and cannot be considered serious, even at that low head.

A puddled core of selected material often has all the advantages of a concrete wall, and in addition does not need to be carried to bed rock. As regards cost, they have a great advantage over any other type, especially in the

isolated regions of the arid west. It will be seen, however, that the function of any core is quite limited and they have in late years been discarded entirely by many Engineers. These dams are built up homogeneously, placing the material in layers with the more select on the up-stream side of the embankment. In this design there is no danger of water collecting between vertical strata, as in the case of a core wall, and causing unequal settlement.

Foundations for an earth fill should preferably be a sandy clay or a loam, and it should never be built on a shale or rock foundation, unless cut off walls are added. Cut off trenches parallel to the axis of the dam, not only in earth but in rock as well and carefully back puddled, add very materially to the strength of the structure.

Attention is called to the ideal location of the outlet from the reservoir. This tunnel, in solid rock, eliminates one of the weakest features of earth dams, namely, a conduit through the fill.

The drawing on page 64 gives the main features and details of this dam as built at an approximate cost of $14.00 per acre ft. of storage. This is slightly in excess of the average cost of 25 other dams in the West, also in India and Spain, which was $13.40.

On page 68 is a schedule of contract costs on this project, as originally submitted by the contractor. It was afterwards found advisable to make several changes in the plans, such as omitting the connections to the distribution system from the feeder canals, omitting a feeder canal from

Basin Cr, also the steel flume on the Birch Creek Canal, and adding a concrete outlet tower and additional cut off walls on the east cliff in the dam.

The wagon roads mentioned, had to be rebuilt because of their submersion by the proposed reservoir. Right of Way includes private and school lands purchased for the reservoir site and the High Line Canal.

SCHEDULE OF PRICES

ON

TWIN FALLS-OAKLEY PROJECT.

BY

FARIS ENGINEERING AND CONSTRUCTION COMPANY

ENGINEERS AND CONTRACTORS.

No.	Description of Article		Cost	Bid
	Dam			
	Clearing and Cleaning Base of Dam		$ 7500.00	
19925	Cu.Yds.Earth Exc.Trenches	.75	14943.75	
875	" " Rock " "	2.50	2187.50	
15775	" " Puddle-back-fill"	.75	11831.25	
5025	" " Loose Rock " "	1.25	6281.25	
7173	" " Core wall "	1.75	12552.75	
6280	" " Concrete Core Wall	12.50	78500.00	
1074181	" " Puddled Embankment	.425	456526.93	
65185	" " Loose Rock Cover	1.25	81481.25	
30000	" " Solid Rock Spillway	10.00	37500.00	
500	" " Concrete "	25.00	5000.00	
883	Ft. Tunnel		22075.00	
2	Sluce Gates-Hoists		9000.00	
				745,379.68
	Goose Creek			
	Clearing and Cleaning			2,500.00
	Gathering System			
9.5	Mi. W.S. Gathering Canal	2450.00	23275.00	
17000	Ft. Steel Flume,Wood Trestle	4.25	12250.00	
	Diverting Weir at Cottonwood Cr.		525.00	
	Spillway at Little Cottonwood Cr.		250.00	
	Connection to Distribution System		500.00	
				96,805.00
	Carried forward			844,684.68

SCHEDULE OF PRICES (CONTINUED)

No.	Description of Article		Cost	Bid
	Brought forward			844684.68
9.5	Mi. E.S. Gathering Canal	2000.00	19000.00	
3100	Ft. Steel Flume,Wood trestle	4.25	17175.00	
	Timber Trestle Mill Gulch		4750.00	
	Diverting weir Basin Cr.		525.00	
	Spillway at Birch Cr.		250.00	
	Diverting Weir Mill Gulch		450.00	
	Connection to High Line Canal		500.00	
				38650.00
	Distribution System			
7.5	Mi. High Line Canal	400.00	3000.00	
500	Ft. Steel Flume,Wood Trestle	2.00	1000.00	
	Diverting Weir, Goose Cr.		3500.00	
	Drop into E. S. Main Canal		350.00	
10	Lateral Gates	20.00	200.00	
				8050.00
15.5	Mi. E. S. Main Canal	1400.00	21700.00	
	Diverting Weir Gates, Goose Cr.		1250.00	
2500	Ft. Concrete Incline	2.50	6250.00	
3	Concrete Drops	525.00	1575.00	
18	Lateral Gates	25.00	450.00	
				31225.00
19	Mi. W. S. Main Canal	1850.00	35150.00	
	Diverting Weir Gates,Goose Cr.		1250.00	
35	Lateral Gates	30.00	1050.00	
34	Concrete Drops	525.00	17850.00	
				55300.00
	Lateral System			
52	Miles Laterals	350.00	18200.00	
216	Measuring Gates	23.75	5130.00	
				23330.00
	Carried forward			1,001,209.68

SCHEDULE OF PRICES (CONTINUED)

No.	Description of Article		Cost	Bid
	Brought forward			1,001,209.68
	Wagon Roads			
	Trapper Cr. Road		3000.00	
	Goose Cr. Road		2000.00	
				5000.00
	Highway Bridges			
20	Main Canal Bridges	150.00	3000.00	
60	Lateral Canal Bridges	25.00	1500.00	
				4500.00
	Right of Way			
760	Acres Private Land	13.82	10500.00	
240	" School "	10.00	2400.00	
	High Line Canal		250.00	
	Damage Flour Mill		5000.00	
				18150.00
	Engineering			
	Surveys, Maps, Plans etc.			25000.00
	Total Estimate			1,054,859.68

CONSTRUCTION.

1. DISTRIBUTION SYSTEM.

Construction on the distribution system of the Oakley Project began in July 1909, with the following organization;- Chief Engineer, surveying parties, and foremen. The Chief Engineer had entire charge of the Construction, supervised the work, made contracts, and directly represented the Company on all matters pertaining to this work. The field parties laid out all ditches, structures, etc., under the direction of the Chief Engineer, and gave general instructions to the foremen. The magnitude and importance of the dam required all of the Company stock and equipment at that place; consequently, practically all of the work on the distribution system was sublet to small private contractors. The term Company, as used here, means the Construction Company or General Contractors on this Project. These sub-contractors were virtually Company foremen, in that they all were paid the same rate on the different classifications of materials in their contracts, and this was fixed by the Engineer for the Company. Also, their pay-rolls were O.K.'d and paid directly by the Company at the same rate as paid for similar labor, teams, etc., on the dam. Each furnished his own stock and equipment, operated his own camp, and was paid from

estimates made by the surveyors of the Company.

After the location of a ditch had been staked out, the ground was cleared of all sage-brush, shad scale, etc., so that the cross sections could be laid out. Grubbing was done largely by hand, using an axe and gathering up the brush with a two-horse hay rake. When the growth was light, a heavy steel rail dragged by 4 horses was capable of removing the growth. The soil was easily plowed by 2 horses and moved by Fresno, Tongue and Slip scrapers. Because of its shape and capacity, also the ease with which it was loaded, the Fresno proved a much more efficient implement than the other two types. Four horses were required to operate them, at a saving of one teamster, and by moving more than 3 times the amount of dirt per trip than the other scrapers did, this type was found to be the most economical. Handling 4 horses was slower than using 2, but considerable time was gained even at that, for only one-third the number of trips were required for the same amount of dirt hauled. As the canals were designed so that the excavation was equal to the embankment as much as possible, there was very little overhaul, and of this, all was paid at the rate of 0.2¢ per 100 ft. after the first 400. Earth excavation and embankment were paid at 12.5¢ per yd., loose rock 75¢, solid rock $1.50, gravel 50¢ and clearing brush $6.00 per acre.

The methods of construction used on the canals were so simple, that they cannot be given any description here, ~~when~~ it may be said that local settlers, entirely inexperienced in this kind of work, were able to take contracts at the above

prices, make substantial profits, and, at the same time, build very workmanlike ditches. Estimates were taken of the work completed at the end of each month, and sub-contractors paid on these figures, deducting 10% until the contract had been fulfilled. Twelve and one-half cents per yd. is an average price paid for such work in this vicinity, although the U. S. R. S. has sublet a great deal of their ditches at a 10¢ rate.

In like manner the railroad and wagon bridges were built by sub-contractors, the Company furnishing all materials. The contract scale of prices was likewise fixed by the Company, in this case being $ 8.50 per 1000 F.B.M. Considering the number of bridges built, their similarity of design, their simple construction, etc., the above rate is high; and $7.00 would have been more nearly the proper scale for this work.

The concrete drops on the West Side Main Canal were built by the Company with one of their foremen in charge. The actual cost of these structures varied from $8.00 to $9.00 per yd. Of this the materials cost nearly $5.00, leaving the balance for labor, excavating, building forms, and mixing concrete. A large part of the expense occurred in hauling materials, sand, gravel, water etc., for considerable distances across the desert. Two-inch form lumber was used, the 8" width giving considerable more satisfaction than the 10" or 12" because it was easier to handle, would not warp, and made less waste when cut up. Each of the 34 drops had, on an average, 50 yds. of concrete, and on construction involving the heavy yardage that this did, a portable mechanical mixer should have

been used. A crew of 8 men will mix by hand an average of 16-18 yds. per day, while with a mixer the output with the same crew will easily be 4 times that amount. It is safe to say that with a proper equipment, the unit cost on these drops would have been considerably less than $7.00 per yd.

2. FEEDER CANALS.

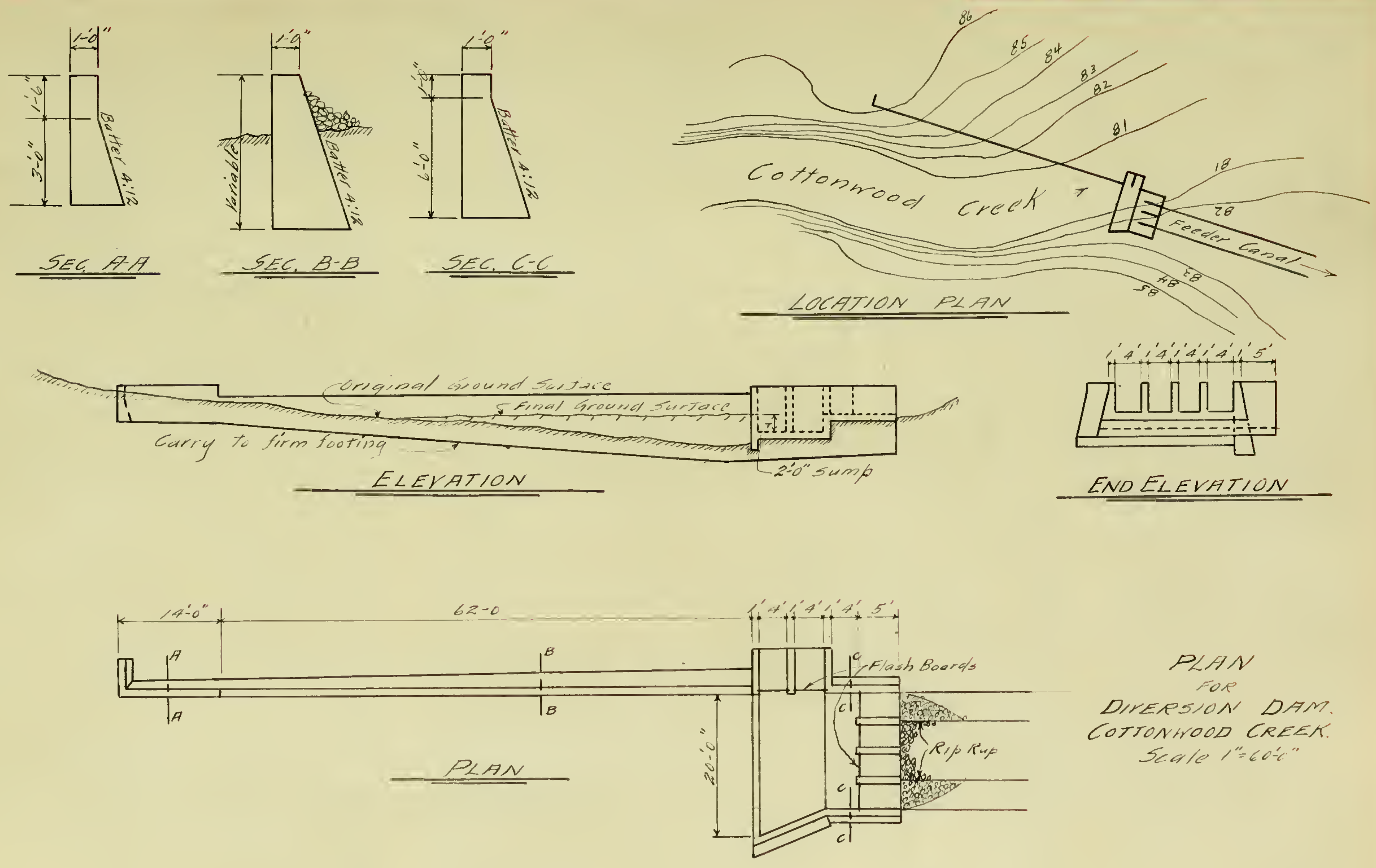
The work on the feeder canals was prosecuted with much the same organization, and by use of the same methods as the distribution system. These ditches were rather hard to build because of the large percent of rock encountered in the excavation, which made considerable shooting necessary. Black powder was found a much more efficient explosive than dynamite, as it acted more slowly, thereby serving a wider area in breaking up the cemented material. Although Fresno scrapers were used to some extent, they did not have much advantage here over the Tongues or Slips because of the difficulty of getting them loaded. It may be said that wheeled scrapers found no part in the construction of any ditch on the project. In the first place they do not hold any larger loads than a Fresno, and again are too heavy to carry loads continually up a steep canal bank. Sub-contractors built a large part of these feeder canals, and their relations to the Company were similar to those existing on the distribution system.

The two tunnels on the Cottonwood feeder canal were also built by sub-contractors. The shorter one, some 350 ft. long, was contracted for at $6.00 per linear ft. for a 4'x6' section; and the other tunnel, approximately 1050 ft. long, was sublet for $7.00 per linear ft. also a 4'x6' section. Under the system used, however, where the Company paid all pay-rolls of the sub-contractor, these unit prices became $12.00 per linear ft. The main reason for this high cost was undoubtedly the low unit cost at which the contracts were let,

that is to say, the mere fact that a contractor would accept such rates, signified that he was incapable from a contract standpoint. A competent miner, with proper equipment would have made a good profit at a rate of $10.00 per linear ft. which also would have meant a saving of nearly $3000 to the Company.

Each tunnel was driven from its two portals, using an 18" gauge track to haul the muck out to the dump. Single Jack, or hand drilling was used principally, although air drills were installed in the long tunnel when it had nearly been completed. Some form of a mechanical drill, either the Chippy or Burley, should unquestionably have been used in driving these tunnels; also a gasoline driven air compressor. Two ft. per shift should have been an absolute minimum in a tunnel of this size. Conditions for rapid progress were favorable in that no timbering was required and no water or gases, other than those from the explosives had to be contended with. The rock excavated varied from a soft, corral colored sandstone to a flint-like, blue lava rock. These tunnels were not lined and had a gradient of .0025, with a 2.0 ft. drop at the upper portals to provide an entrance head for incoming water from the wider ditch above.

The diversion dam at Cottonwood creek was built by a sub-contractor on a force account basis. Gravel and water were taken from the creek nearby, and consequently the cost of this concrete structure was $7.25 per yd. The 110 yds. of concrete necessary here were mixed by hand. By building the sump first, the creek was then diverted through it, while


1'-0"
1'-6"
3'-0"
Batter 4:12
SEC. A-A
1'-0"
Variable
Batter 4:12
SEC. B-B
1'-0"
1'-0"
6'-0"
Batter 4:12
SEC. C-C
86
85
84
83
82
81
18
28
38
48
58
Cottonwood Creek
Feeder Canal
LOCATION PLAN
1' 4' 1' 4' 1' 4' 1' 4' 1' 5'
END ELEVATION
Original Ground Surface
Final Ground Surface
Carry to firm footing
2'0" sump
ELEVATION
14'-0"
62-0
1' 4' 1' 4' 1' 4' 5'
A
A
B
B
C
C
C
C
Flash Boards
Rip Rap
20'-0"
PLAN

PLAN
FOR
DIVERSION DAM.
COTTONWOOD CREEK.
Scale 1"=60'-0"

the remainder of the dam was under construction. There were no difficult problems involved, as the work was done during the low stage of the creek, possibly 10 sec. ft. flowing at that time. See drawing, page 77 for a plan of this diversion dam. The diversion built at Birch Creek was essentially similar to that described above.

Over 200,000 F.B.M. were found to be necessary in the construction of flume trestles, which varied in height from 96 ft. at Wagon Road Gulch to 8-10 ft. where it contoured around the steep side hills in Cottonwood and Goose Creek Canyons. These flumes served as a striking example or illustration of the advantage in employing skilled men in one line of work, over the other type, which are all-round men. The flumes at Cottonwood were built by a gang under an ordinary carpenter foreman, while those at Goose Creek were erected by an experienced flume carpenter on a much higher salary. In comparing the costs of erection, it was found that the Cottonwood structure cost $2.50 more per M, which would have paid the difference in wages given the two foremen, more than twice over.

The posts of the flume trestles rest on concrete pedestals, one ft. square on the top and having a 0.5 to 1 batter on all four faces. Two heights were used, namely - 6" on solid rock foundation, and 18" on earth. The cost of these piers, which were built by the Company, are given on the following table. Even considering their very unfavorable location, and the difficulty of reaching them with construction materials, etc., these costs are high. Under a strict contract

basis, with competent labor, these costs would have been nearer 85% of that given below.

COSTS - CONCRETE PEDESTALS FOR FLUMES.

Labor \$2.50 Foreman \$3.50 Hand Mixing of Concrete.

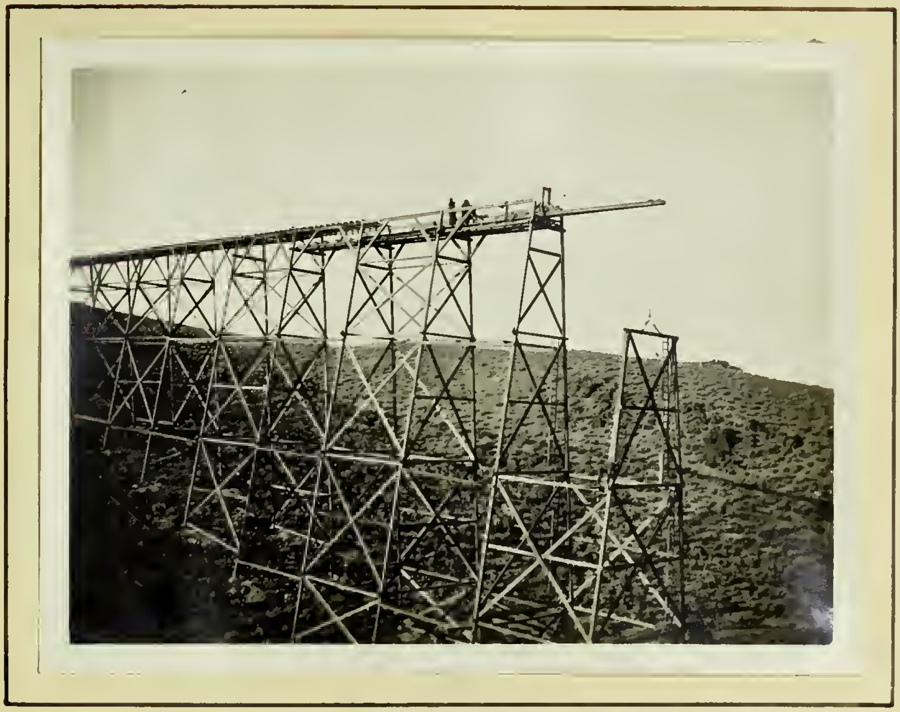
No. and Size	102-18"	8-6"
Excavation	\$ 48.10	\$ 3.76
Moving plant, Runaways, etc.	10.20	.80
Placing Forms	24.50	1.92
Concreteing (Labor)	106.20	1.60
Sand @ \$2.25	52.30	.76
Water	8.40	.12
Cement @ .90	90.00	1.24
Forms (Lumber)	5.10	.40
	334.80	10.60
	3.37 Each	1.32 Each

All 6" piers required 3/4"x12" anchor spuds into the rock and the drilling necessary here is included under the head of excavation.

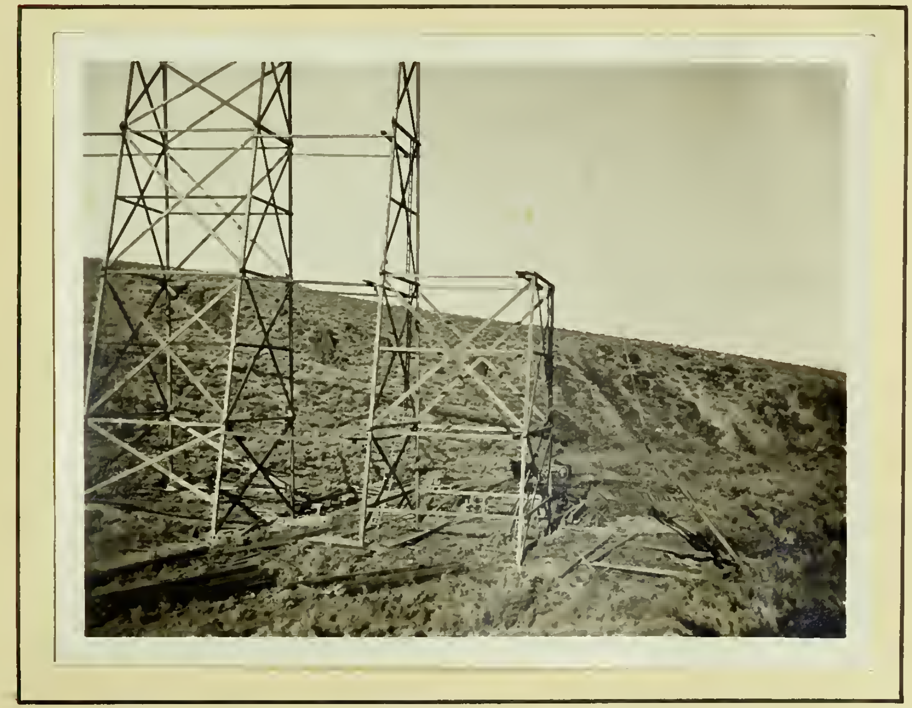
The following refers to the Goose Cr. end of the work which the writer had charge of personally. A description of the design of the flume trestles is given in the article on "Desgin of Feeder Canals". On low work no especial methods were employed in construction, except that templets were used entirely, each man keeping at one kind of work as much as possible. Cost records on the low structures gave from \$14.00 to \$16.00 per 1000 F.B.M.

It was found impracticable to erect the high trestle, deck by deck, on account of the long span between bents, also the great amount of false work that would be necessary, if this method was used. The writer then designed a cantilever erecting beam which operated on a temporary platform 4 ft. below the top stringers. In this way, the trestle was built complete as it advanced from the north side of the canyon. The erecting beam, equipped with ropes and windlass, hoisted each section of a bent to its place. The upper illustration on page 81 shows the cantilever beam picking up a section of a bent from the ground where it had been cut and framed, and the lower illustration on the same page, shows the same bent-section in place on the cripple cap of the third deck. One end of the 3"x6" girts was fastened by a bolt to the bent post just below the cap, before the section was raised, and when the latter had been set in its final position, the other end of these girts would then be swung up and nailed into place on the bent last erected. Bents forming a tower were sway-braced together immediately. Light ladders, fastened to the caps of each bent-section by hooks, were used by loft men in fastening the bracing. As the traveler advanced, the temporary platform was torn up and the flume carriers placed in position on the stringers. After the structure was completed, the temporary girts between towers were stripped off.

Good time was made at this erection, considering the inexperienced loft men employed. Based on the record made later with the same crew, on a similar though much lower trestle, the unit cost given below could have been reduced



COTTONWOOD FLUME TRESTLE.
(Showing Contilever Erecting Beam.)



COTTONWOOD FLUME TRESTLE.
(Maximum Height 96 ft.)

5 to 8%. The erecting beam is highly adapted to this type of work, being rapid and safe, also employing a minimum amount of false work. No.-1 rough Oregon fir lumber was used throughout and all designs made according to unit stresses recommended by C. Shaler Smith. (See Trautwine.)

One ~~great~~ objection to this design is that the structure is nailed throughout. High trestles, like this one, are bound to sway considerably in the wind, and consequently the nails are sure to work loose.

COSTS - HIGH FLUME TRESTLE.

546' long. 96' high (max.) 27.9 M - F.B.M.

Labor	775 hrs.	@.25	$ 193.75
Carpenters	649 "	@.35	227.15
Foreman	148 "	@.60	88.80
Mule	150 "	@.10	15.00
			524.70
			$ 18.80 per M.

Lumber $24.00 per M. F.O.B. Oakley, Idaho.

The Lennon steel corrugated flume #120 is used. It comes in 2 ft. widths and when erected, the end corrugations overlap, being drawn up under the 4"x4" carriers by 1/2" rods that lie in the saddle of these corrugations. On curves, a special form is used, two sections being soldered together in the factory to form the required angle. Hanging this flume in place cost about 8¢ per linear ft. In painting the trestle, 1 gal. of mineral paint, covered 100 sq. ft. of the rough

surface at a labor cost of 40¢ per gal.

STEAM SHOVEL PIT. (1912)

OAKLEY DAM. AUG. 1912.
(80% Done.)

3. THE DAM.

The personnel at the dam consisted of a Chief Engineer, Assistant Engineer of Construction, Superintendent, Assistant Superintendent, Master Mechanic, Chief Electrician, Time-keeper and Foremen. The Chief Engineer represented the Company in all matters and directed the construction policies in the field. The Asst. Engr. assisted in the designs of structures both temporary and permanent, laid out the work in the field, and issued orders to foremen regarding the manner of its construction, subject to the approval of the Chief Engr. The Supt. had complete charge of the rest of the working force at the dam, directed his foremen, hired and discharged men, and was responsible to the Ch. Engr. for the proper progress of the work. The Asst. Supt. acting under the direction of the Supt. had charge of the working force on the night shift. The Master Mechanic, Chief Electrician, Time-keeper and Storekeeper had charge of their respective Departments, subject to the direction of the Supt. By store is meant the Company Commissary, where merchandise, cook house supplies, tobacco, etc. were sold.

The most striking feature of the above organization is the absence of either a material clerk or any one keeping cost data. No attempt was made to keep track of where supplies went, neither were any accurate costs kept of the different kinds of work. Foremen handed in each night a card showing the number of men he had that day, and the nature of the work

done; but these cards were often incomplete, even missing at times, and a record of the amount of work done seldom appeared, ~~and~~ this was all that could be expecfed, for some of the best foremen were unable to write properly. Materials and Costs, two of the most vital factors in contracting should have their own respective departments, the benefits derived from such information more than paying the expense incurred.

Any cost data that appears on these pages was taken and computed by the writer directly or under his direction, except that prior to April 1911. This data was computed from the foremen's cards and consequently is only approximate.

Construction work on the dam began about the same time that it did on the distribution system - namely July 1909. The dam is located in the mouth of Goose Creek Canyon. At the east end the cliff stands almost vertical, while at the other end there is a much more gentle slope, with bed rock coming to the surface. The base had a heavy growth of weeds, sage brush, willows, etc., that were cleared off immediately and the earth plowed into furrows, to make a good bond for the fill placed on it. The core-wall trench was begun first at the west end and carried to bed rock. In the creek bottom this was found at a maximum depth of 27 ft. below the surface. The soils encountered were strata of sand, gravel and boulders, with about 15 ft. of loam at the surface. A 3 ft. concrete wall was then built in the trench up to 9 ft. above the natural ground of the canyon. Upon it the 1 ft. reinforced concrete core wall rested. The trench was immediately back puddled with selected material on either side of the wall.

Half-way between the core wall and the upper toe, a cut off trench was dug 10'x8', in rock section and a 10 ft. base with 1 to 1 slopes in earth. In the flat creek bottom, this trench was omitted. It, likewise, was back puddled with selected material.

At the upper toe another cut off trench was dug, this time to bed rock with a 10 ft. base and 1/2 to 1 slopes, the rock sections at either end being 10'x8'. Where this trench was dug in earth, a small concrete core 18" thick and 3 ft. high was built in the bottom of it and then back puddled with selected material.

To provide for drainage on the down stream side of the dam, a trench 10'x6' in section was dug along the entire toe, and back filled with loose rock. The addition of an intermediate trench and connected to the one at the lower toe by means of several short trenches, would aid this drainage considerably in the event of a heavy seepage.

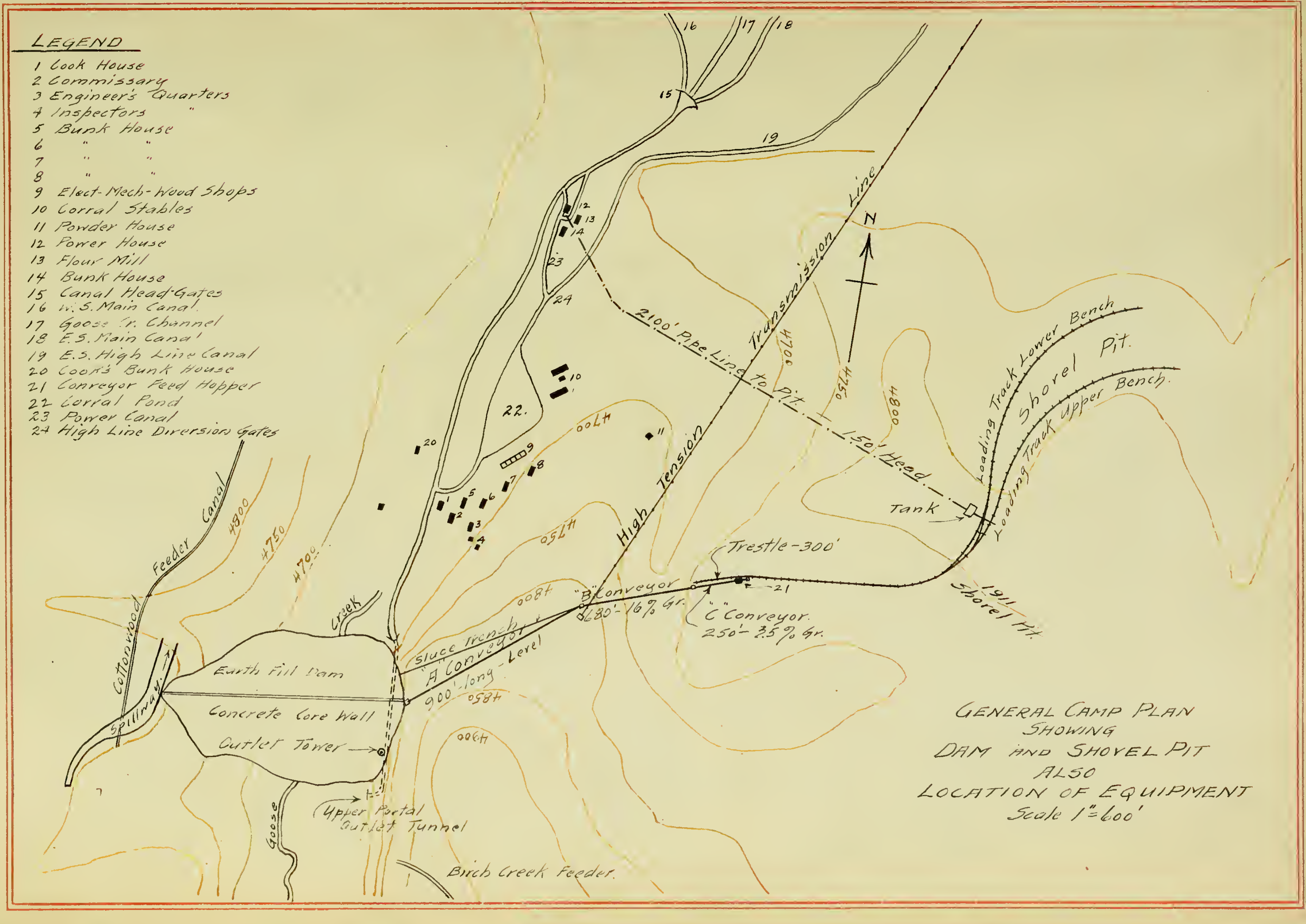
The original plans called for a trench at the upper toe similar to the intermediate cut off trench, but after encountering a large amount of sand, boulders and water in excavating the core wall trench, it was decided to carry the upper trench to bed rock and put in a small concrete core wall, as described above. The stratification of material was very marked at these trenches carried to bed rock, being nearly horizontal. Water seeped through them freely and required the service of 2-3" and 1-5" suction pumps to keep it out sufficiently for men to work in them. No sheeting was required however, except in places where the sand caved in.

Derricks and skips were used in this excavation, being loaded by hand, and hoisted by teams, while wheeled scrapers carried the material away. This method proved quite satisfactory and good time was made considering the materials encountered, especially the boulders, many of which were quite large.

The east cliff is a disintegrated lava rock, having a great many crevices and fissures in it. To guard against seepage through them, the cliff was shot off some 15-20 ft. back, to give a solid bond for the concrete core wall. As an additional guard against seepage, 3 vertical cut off walls of concrete were built on the face of the cliff, about 20 ft. apart and up-stream from the core wall. These walls are 18"x5', and follow the cliff up to the dirt slope of the dam. They are anchored to the cliff by 7/8" sq. twisted Ransome bars, grouted and wedged on 3 ft. centers. Like the core wall and rip rap, these walls are built up as the fill advances. All loose rock and silt were cleaned out of the other crevices as much as possible and back puddled with selected material.

The main outlet tunnel, 870 ft. long and 8x10 ft. in section, was begun at the same time the trenches were. It was driven from four faces and good progress was made at the rate of 3.5 ft. per shift. The rock was quite soft, having a great many seams in it, which made the drilling and shooting quite easy. The tunnel was not lined except about 100 ft. at the upper portal and 50 ft. at the lower tunnel. Considerable rock has caved in since then, and it will require a heavy lining to prevent this from continuing in the future.

In April 1910, Goose Creek was diverted through the



GENERAL CAMP PLAN
SHOWING
DAM AND SHOVEL PIT
ALSO
LOCATION OF EQUIPMENT
Scale 1" = 600'

LEGEND
1 Cook House
2 Commissary
3 Engineer's Quarters
4 Inspectors "
5 Bunk House
6 " "
7 " "
8 " "
9 Elect-Mech-Wood Shops
10 Corral Stables
11 Powder House
12 Power House
13 Flour Mill
14 Bunk House
15 Canal Head-Gates
16 W. S. Main Canal.
17 Goose Cr. Channel
18 E. S. Main Canal
19 E. S. High Line Canal
20 Cook's Bunk House
21 Conveyor Feed Hopper
22 Corral Pond
23 Power Canal
24 High Line Diversion Gates

16
17
18
15
19
12
13
14
23
24
22.
10
20
9
8
1
5
6
7
2
3
4
11
N
4700
4750
4800
4850
4900
High Tension Transmission Line
2100' Pipe Line to Pit - 150' Head
Tank
Loading Track Lower Bench
Shovel Pit.
Loading Track Upper Bench.
1911 Shovel Pit.
Trestle-300'
21
"B" Conveyor
680'-16% Gr.
"C" Conveyor.
250'-3.5% Gr.
Sluice Trench
"A" Conveyor
900' long - Level
Creek
Earth Fill Dam
Concrete Core Wall
Outlet Tower
Upper Portal
Outlet Tunnel
Goose
Birch Creek Feeder.
Cottonwood Feeder Canal
Spillway.

tunnel which allowed the core wall and cut off trenches to be dug across the old channel and the 3 ft. core wall completed well up on the east cliff.

The original plans were to sluce the earth fill into place by using water from the Birch Creek Feeder Canal, but legal difficulties over water rights, made this impossible; and consequently water had to be pumped up from Goose Creek for this purpose, against a 170 ft. head. A Goulds #6 Rotary Pump was used, it having a capacity of 675 gal. per minute at this head. Large Centrifugal pumps were also tried but found to be entirely inefficient. Single stage pumps of this type will not operate successfully against such a head. The water merely churns in the shell at the ordinary R.P.M. which they are designed for, and if speeded up, will break the shell.

Preparations on the dam were made by building up the lower toe about 20 ft. using wagons and scrapers. This allowed the finer slucing material to run toward the upper toe. Material was obtained from a pit near the one used in 1911 and hauled by cars and the B. Conveyer to the slucing flumes. These flumes were built on trestles and had an original grade of 4%, a 20" base and were lined in the bottom with Cottonwood blocks on end. See Fig. 5 page 109. Later this was changed to 10% grade, same section but lined with sheet steel.

The slucing method proved a failure, chiefly because of the lack of water. Data obtained from successful slucing operations elsewhere show that an average stream of 3.5 sec. ft. is required to sluce 10,000 yds. per month (flume grades, 5 to 12%; material, gravel and earth.) Here,

75,000 yds. were being attempted with 1.5 sec. ft. stream. Black sand caused considerable trouble also, by choking up the flumes. Approximate data on the cost of slucing gives 12¢ per yard, and that for hauling in dirt on the lower toe, using wagons, slips and trap, 20¢ per yard.

On the dam it was found that considerable stratification of material resulted from the slucing operations. This should condemn the method on very high structures, and where conditions are favorable for slucing the slopes of the dam, should be made flatter than usual. Slucing was then abandoned and preparations made for building the dam by the Mechanical Method - vis., placing the material dry in layers, then sprinkling and rolling it.

No borrow pits were found near by which were high enough to allow dirt trains to run to the east cliff over the dam. There were several small pits in the gulches west of the spillway, but the material here was a fine lava ash, fit only for puddling, etc. Suitable earth was found about 3/4 mile east of the dam. See page 89. It was a cemented gravel, containing a heavy percentage of earth, and very hard to excavate. The upper illustration on page 105 shows the 1911 arrangements at the borrow pit, the terminus of the Dinky track, and the loading hopper for the B. Conveyer.

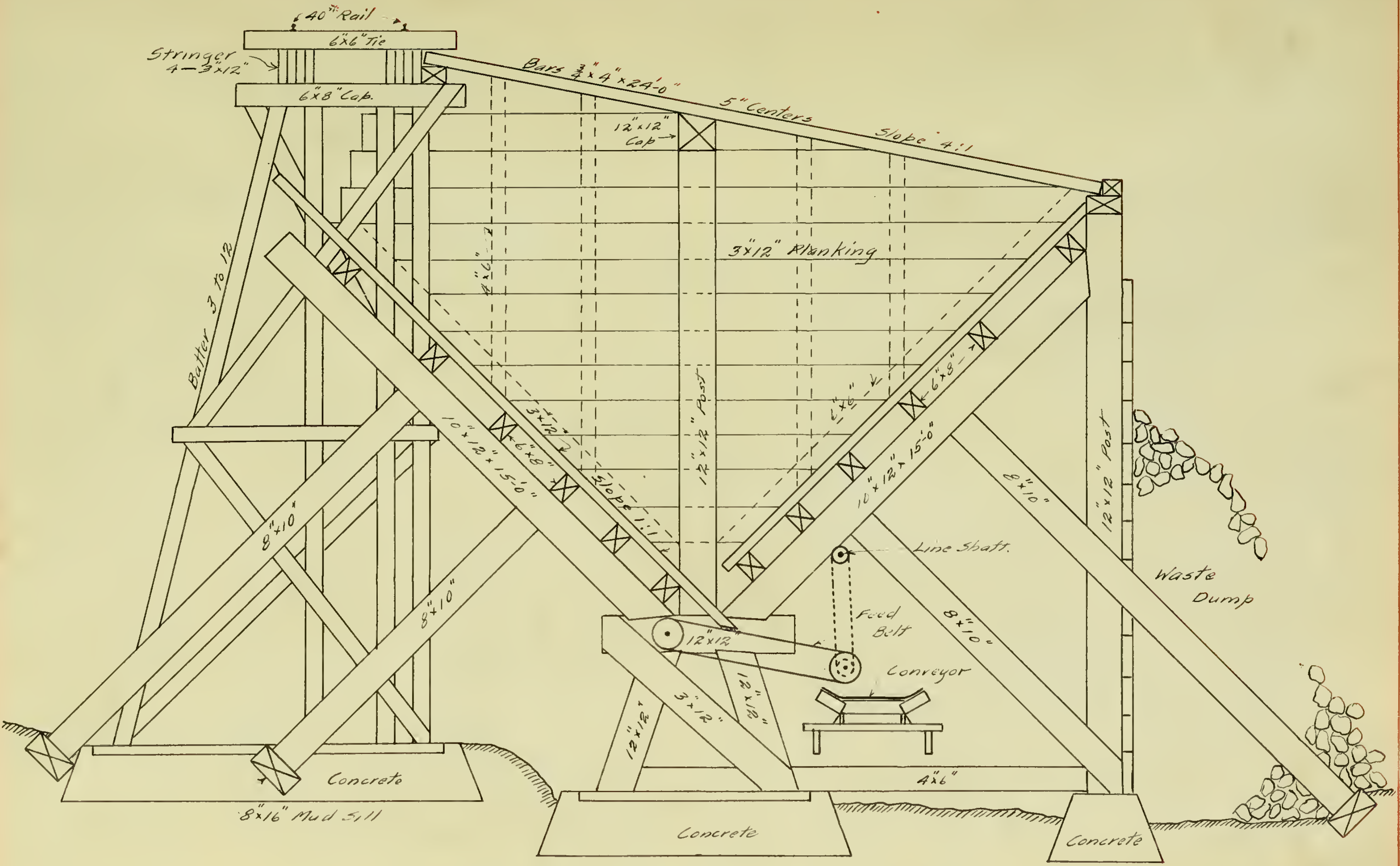
The following equipment was used in the pit to handle the material. One #60 Marion Steam Shovel, 2 - 16T Davenport Dinkies, and 35- 4 yd. 2 way Western Dump Cars. Twenty pound steel rails were used on loading tracks and main line and found to require constant surfacing. Two

15-car trains were operated, one being in transit while the other was loading. Under ordinary weather conditions a Dinky could pull a loaded train up a 2% grade and the empties up a 6% grade.

The Dinky trestle at the loading hopper was designed for a live load of one 16T Dinky on a 60" wheel base. The bents were on 8'-0" centers and had four posts, these being poles having a minimum diameter of 6". On page 93 is shown a drawing of the trestle used in 1912, which was similar to the 1911 structure. It was built on a 40 degree curve, the outside bent posts, except at the hopper, having a batter of 3 in 12.

By referring to the map on page 89 it will be seen that the land adjacent to the dam site is very rough, having many deep gulches. Miles of contour track would have been necessary to bring cars from the shovel pit to the dam; and if bridges were built across the gulches, the grades would have been inaccessible for the Dinkies. This led to the adoption of a conveyer system. The B. Conveyer had been built the year previous, to haul earth for the slucing process, and in the winter of 1910-11 the A. Conveyer was erected. The former had a grade of 16% and is 680 ft. long, while the latter was 900 ft. long and level. They were built on trestles whose maximum height was 68 ft. at Mill Gulch. (B Conveyer)

In general, the bents had 2 posts, were on 24 ft. centers and the stringers on 42 inch centers. The head drive and tail drum frames were anchored in a concrete base and set on solid rock. The belts were of rubber and 36 inches


40# Rail
6"x6" Tie
Stringer 4—3"x12"
6"x8" Cap.
Bars 3/4"x4"x24'-0"
5" Centers
Slope 4:1
12"x12" Cap
3"x12" Planking
Batter 3 to 12
4"x6"
3"x12"
6"x8"
10"x12"x15'-0"
Slope 1:1
12"x12" Post
6"x6"
6"x8"
10"x12"x15'-0"
8"x10"
12"x12" Post
8"x10"
8"x10"
Line Shaft.
Feed Belt
Conveyor
12"x12"
12"x12"
3"x12"
12"x12"
8"x10"
Waste Dump
4"x6"
Concrete
8"x16" Mud Sill
Concrete
Concrete

SECTIONAL ELEVATION LOADING HOPPER

wide, 6 ply on the B. Conveyer and 7 ply on the other. When the dirt became wet in rainy weather it was very hard to handle, in the chutes at the dam. To avoid this as much as possible and as a protection for the rubber belts against the sun and wind, the conveyers were housed. To take up the slack in the belts, gravity idlers, operating in a vertically slotted frame, were used. The idlers were of cast iron, 18 inches in diameter and raised or lowered in their frames, according to the slack in the belts. On Conveyer A, which was extremely long, auxilliary weights had to be added to the idler to keep the belts from slipping on the drive drums. Carrier and troughing rollers were spaced on 5 ft. centers, and return carrier rollers on 20 ft. centers. The belts were run on a speed of 250-300 ft. per minute and the average haul during the season was 45,000 yds. per month.

On page 93 is shown an elevation of the feed hopper used in 1912. The 1911 structure embodied practically the same principles, although it was much smaller. The hopper set directly over the conveyer belt, and dinky trains reached it by means of the trestle. A grid of parallel bars on 5 inch centers, with a slope of 8 to 1 down from the track, covered it. The bars were ¾ x 4 inch in section and 24'-0" long. This grid separated all rock and lumps of cemented gravel, which could not be broken up, and they were wasted on a nearby dump.

The openings under the hopper were 1 ft. by 4 ft. wide, with no doors. Feed belts of 32 oz. canvas, 8 ft. wide revolved under these openings, caught the material and then

dropped it on the conveyer belt. The belts were independent of each other and operated by chain drives and friction clutches, from a live shaft to the tail drum of the conveyer. When operating, the speed of the feed belts were one-tenth that of the conveyer. Slats, spaced every 2 ft., kept the canvas belt from wrinkling. Under ordinary use, it would last for half a season. The only advantage of a number of feed belts was that the hopper could then be more easily emptied, as the loose material stood almost vertical in the bins, around the openings.

The following equipment was installed to operate the conveyers. Conveyer A. - 1-50 H.P., 550 V, D. C. Motor, (Gen. Elec. Co.). Conveyer B. - 1-65 H.P., 550 V, D.C. Motor, (Gen. Elec. Co.), At first, these motors were attached to the conveyer drives by gears and pinions, but the excessive vibration caused considerable trouble, expecially in overheating the motors; and therefore, belt drives were substituted, which gave good satisfaction.

Current was generated at the Company power house, (see map on page 89) with the following equipment:

1 - 39" Victor Vertical Water Wheel (13 ft. head)
1 -160 H.P. Ball Engine. Simple high speed, Non-Condensing.
2 - 80 H.P. Boilers
1 -100 K.W. Edison Bi-Polar Generator, 550 V.
1 -700 V. Ft. Wayne D.C. Voltmeter
1 -800 A. Ft. Wayne D.C. Ammeter
2 -G.E. Carbon type Circuit Breakers

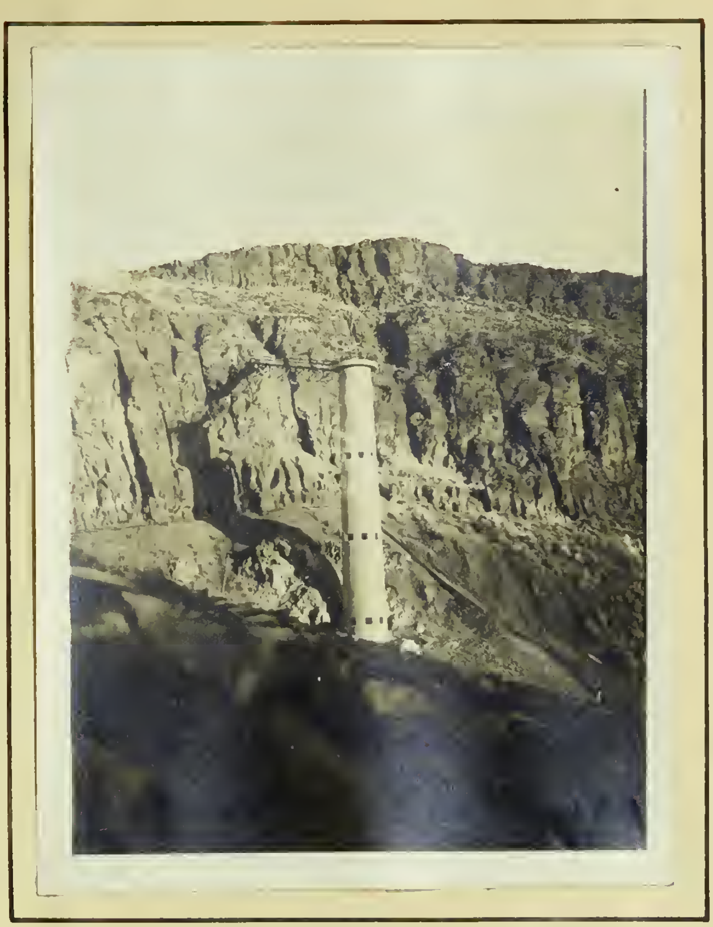
1 - 400 A. Westinghouse Ammeter

1 - 250 A. Triple pole Switch board.

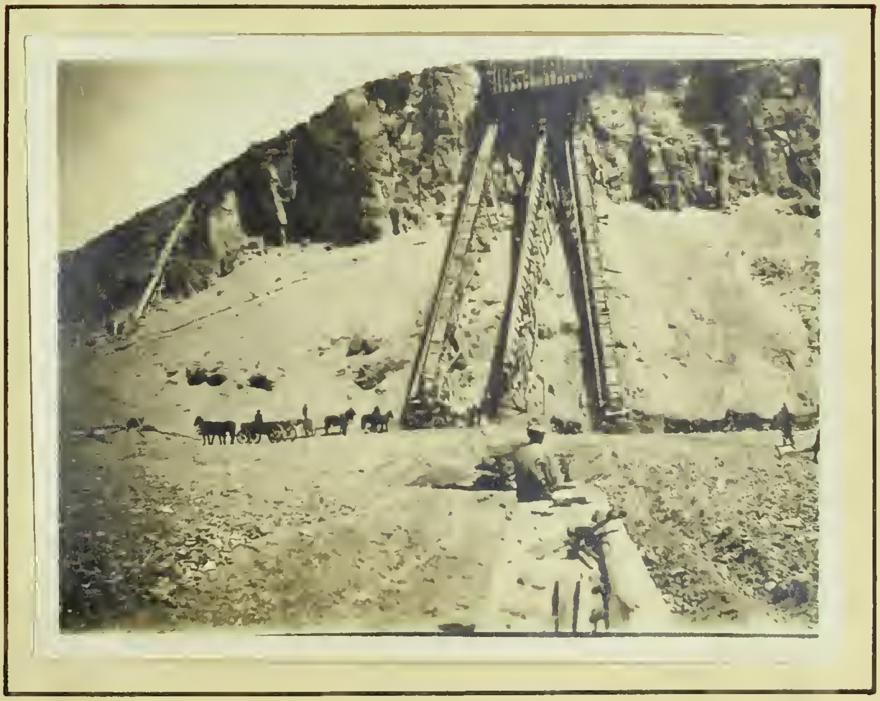
On an average about 90-95 K.W. were developed here during the season of 1911, at a cost of $125 - $135 per day of 24 hrs. With no graphic wattmeter, it was impossible to obtain accurate costs per K.W. Hour. The turbine, when using all the water available, developed 50 H.P. and was hitched to the same main shaft as the Ball Engine.

An auxilliary Hydro-Electric plant on Trapper Creek developed 25-30 K.W. and was operated in parallel with the main power house. The equipment consisted of 1 Leffel, style 18-15 1/4 inch D.D., Horizontal Turbine operating under a head of 80 ft. and 1-35 K.W. Fairbank's Morse Generator, 550 V. The output was sufficient to furnish all lights for the camp, dam and shovel pit, also run the rock crusher and concrete mixer, alternately.

On the east cliff under the head drive of the A. Conveyer, and on the center line of the dam, a storage bin was built with 3 chutes leading from it. The bin had a capacity of 200 yds. and was designed to provide for the fluctuations in the conveyer haul. The chutes were built on trestles and had a slope of 45 degrees. Figure 1, page 109 shows a section of these chutes as originally built, being covered by doors and lined in the bottom. The gate controls were located at the lower end of the chutes, and wagons loaded directly from them. This proved very unsatisfactory as the material packed and choked when the gates closed. It would not run smoothly, but moved in sections, one section making a heavy impact



OUTLET GATE TOWER.



STORAGE BIN AND CHUTES (1911)

against the section below, when it suddenly released and slid down. Another chute of section, as shown in figure 2 page 109 was then built on a slope of 1 1/4 to 1, but it too, had the same faults, only to a lesser degree. The gates were then placed at the upper end of the chutes and the material allowed to run wild into the wagons below. This had to be discontinued, because rocks coming down a distance of 150 ft. had a sufficient velocity to drive them nearly through the dump wagons.

A portable loading hopper with a capacity of eight wagon loads, was then built and placed under the lower end of a chute. Wagons came under it and were loaded, and when running steadily, 4-1.5 yd. dump wagons were loaded per minute, one man operating the feed door of the hopper, and another operating the chute gates up under the storage bin - kept the portable hopper full. This method proved very satisfactory and as the fill raised, the chutes were cut off and the loading hopper likewise raised.

From the portable hopper the material was hauled out and dumped in wind rows, parallel to the core wall. Plowing lengthwise and using Tongue scrapers, these wind rows were leveled out to 6" layers. Water was then sprinkled on it, and later rolled with a 16T Avery traction engine. The rear wheels soon wore out running over the loose uneven fill, and were replaced by Marion Steam Shovel wheels, 24 inches wide and each weighing 3000 pounds. The fill was laid on a $1\frac{1}{2}\%$ grade down from the loading hopper towards the west end of the dam. This was done to help the stock in hauling

loaded wagons, 12 to 16 teams being used per shift. The fill was sluced into place on either side of the core wall, also where it joins the cliff at the east end and the natural earth surface at the west end of the dam. The material was ideal for dam construction. After sprinkling and rolling, it became very compact, due to the cementing qualities of the earth. From experiments it was found that no perceptible settlement or shrinkage occurred during the following winter. Based on the car count in the pit as compared to the cross sectional estimates at the dam, a shrinkage of 10 to 12% occurred in the process of sprinkling and rolling as against 6% for puddling.

The upper toe was built of a selected material for a distance of 20 ft. back from the face. This earth was largely the run of finer material from the shovel pit, also that taken from the pits beyond the west end of the dam. All rock larger than 2" in diameter was removed from this 20 ft. zone.

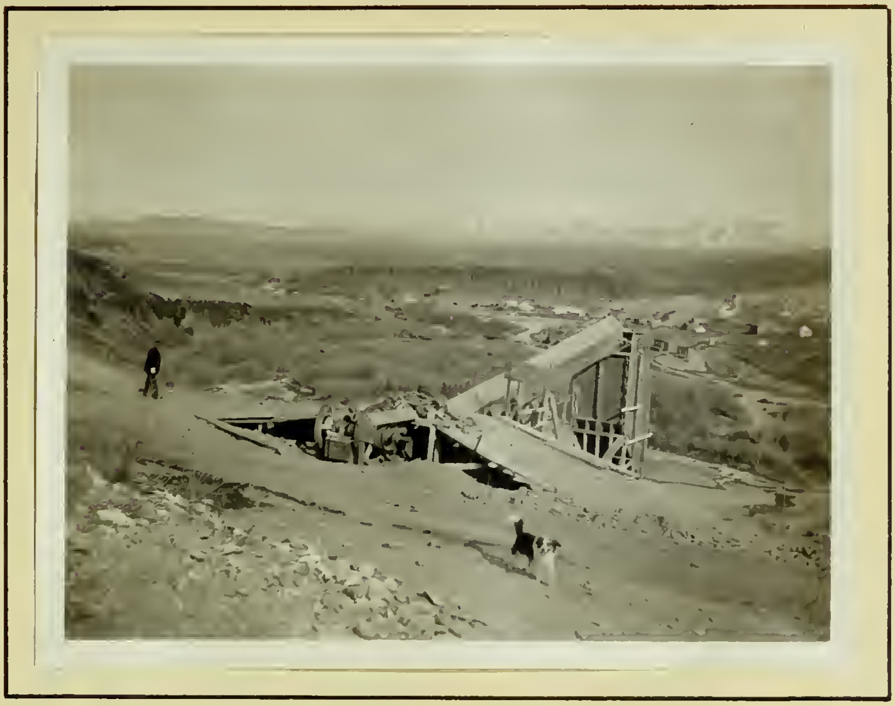
Following the fill as it raised, the rip rap was placed by hand, 3 ft. thick on the upstream face and 2 ft. thick on the downstream side. The stone used was from ¼ to 1 cu. ft. in size, except a layer 1 ft. thick next to the earth on the 3:1 slope, which was much smaller, being the size of coarse crushed rock. The rock was taken largely from the spillway excavation, although a considerable amount was shot down from the east cliff, where it fell on the fill and was easily placed. A three foot thickness of rock on the upstream slope is more than used ordinarily on earth dams, but

considering the extreme high winds in this canyon it was deemed advisable to take this extra precaution against erosion by wave action.

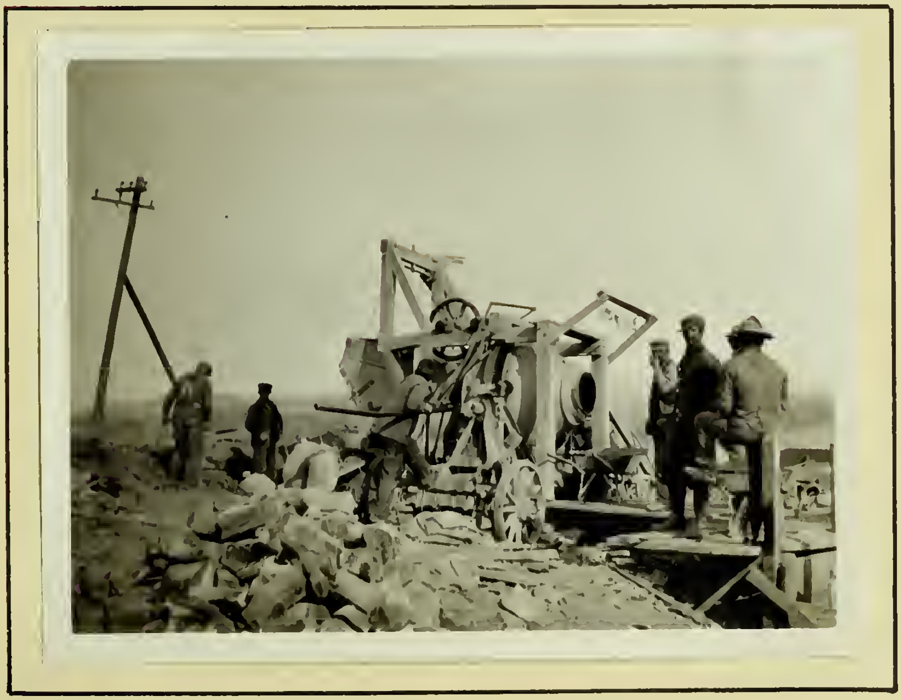
The pump for the sprinkling system was located at the lower portal of the outlet tunnel. It was a 2 stage, 3" American, pumping 100 gal. per min. against a 230 ft. head (static plus friction), and requiring 25 K.W. to operate it. A 3" main ran from the pump up over the east cliff opposite the dam. Here, 3-2" pipe lines took out from it, and were laid in the fill parallel to the axis of the dam. Two inch stand pipes were erected, 100 ft. apart on each pipe line and as the fill raised, 3 ft. sections were added. Using a short hose and nozzle, the entire surface of the dam was easily reached from this piping system. Considerable trouble was experienced with pipes breaking in the fill. There being no appreciable settlement in the dam, and great care having been taken to make all sleeve and union joints as tight as possible, no satisfactory explanation for this trouble was ever given. By grouting a batch of rich concrete mortar around each sleeve, considerable less breaking occurred which indicated that at least a part of the trouble was located there. Trouble was also encountered with the centrifugal pumps. In order to pump 100 gal. per min., its speed had to be increased 25% over its normal rating. This was the cause of a number of shell casings being broken during the season. At least a 3-Stage pump should have been used here. The substitution of 3" pipe lines in the dam would also have materially lessened the strain on the pump.

As stated before, the capacity of the sprinkling pump was 100 gal. per min. This was not sufficient to sprinkle the fill and sluice along the core wall, etc., thus instead of 15%, the sprinkling alone was hardly ever over 5%. In the construction of earth dams by the Mechanical Method, as much water should be used as possible and still not interfere with the wagon haul and the rolling of the material. It is a question whether this amount would ever reach 15%, probably 10% being a much more average value.

The concrete core was built up in lifts as the fill raised. These lifts were 34 inches, or the width of the woven reinforcing material. The reinforcing used was the American Wire Co.'s #9 mesh reinforcing wire, butt joined and wired together. Wooden forms, in 16 ft. sections, of 1 inch lumber with 2"x4" ribs every 2 ft. were used. They were held in place, by means of wires, which drew them in tight against the wall just built and above by short braces to the fill, a 1 ft. spreader holding them apart. Hand mixing was used entirely, the dam being so wide that little core was built, proportionately. Sand and crushed rock were used in proportions which gave as dense and impervious a concrete as possible. A crusher plant operated in connection with the spillway excavations, crushed all rock that was too small to be used as rip rap. On page 102 is shown an illustration of this plant, which was designed by the writer. Loaded wagons were driven upon the platform and dumped in front of the crusher. One man easily fed the material while a conveyer belt carried the crushed product up a 20% grade to the 20 yd. bin. Here it was



ROCK CRUSHER PLANT.



CONCRETE MIXER PLANT.
(Self Charging Skip)

easily loaded into wagons, which drove up under a chute on its lower side. A 30 H.P. 440 V., A.C. motor was used to operate the crusher and run the conveyer. The following is a table of costs on crushing rock by this plant. The assumption was made that 1 yd. of loose rock would make 1 yd. of crushed rock and which is true provided the rock does not materially change its shape, but rather its size.

COSTS - CRUSHING ROCK.

63 Yds. 250 ft. haul.

3 Men loading wagon	$ 7.50		
1 Man feeding crusher	2.50		
1 Man & Team	5.00		
Interest & Depreciation on Complete Equipment of Plant	2.00		
Current	2.00		
	$ 19.00		
		30¢	
(see below) Drilling & shooting rock		13¢	
		43¢	per yd.

COSTS - SHOOTING SOLID ROCK

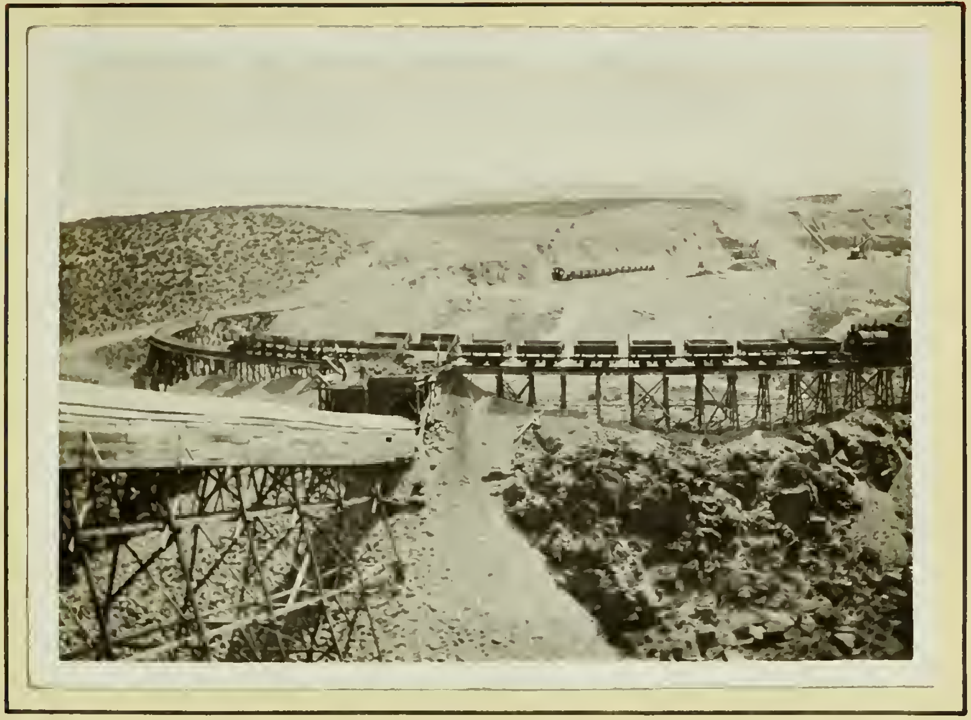
912 Yds. -- Some Seams.

Drilling 100 ft. @ 35¢	$ 35.00	
Powder & Dynamite	85.60	
	120.60	13¢ per yd.

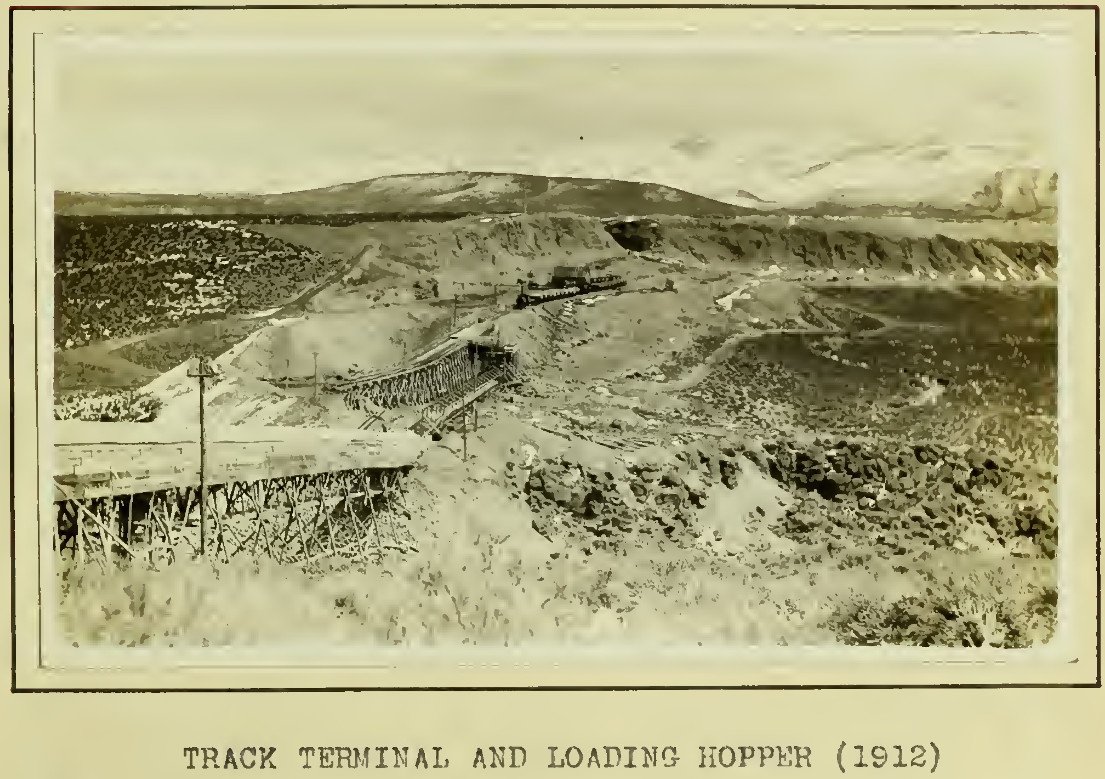
In reviewing the seasons work it was easily seen that two great faults existed, namely in the shovel pit and at the power house. The material was very hard to excavate and the cost of shooting it became prohibitive. The shovel in cutting into the bank gradually increased the face until it was 60 ft. high at the end of the season. This was very dangerous, heavy slides occurring at times. Shooting the bank cost as much as 9¢ per yd., churn drilling averaging about 30-35¢ per linear ft. Because of the high face, it was impossible to shoot down the bluff without the material sluffing on the loading tracks, and this often caused long delays. Unquestionably "Coyote" holes should have been used in shooting a bank as high as this one, for they would have had from 3 to 4 times more range than a row of holes drilled from the top, and compared to the high cost of drilling, would have been more economical.

It was then decided to abandon this pit in 1912, and cut through the hill to the north slope of the same bank, which seemed to be softer. By using two shovels, each working on separate benches, a 20 ft. face would be the maximum, and also allow them to work slower and at the same time break up the material, thus eliminating the expense of shooting.

The power house also was the cause of many delays which was due chiefly, to the inadequate equipment. In the first place, it was all second-hand machinery which was not capable of its rated output; and second, no data was available as to the horse power to be required by the conveyers,



TRACK TERMINAL AND LOADING HOPPER (1911)



TRACK TERMINAL AND LOADING HOPPER (1912)

and in assuming this, it was considerably underestimated. Instead of adding to the plant it was deemed advisable to contract for current from some nearby Hydro Electric Company.

The Dinky trestle on the 40 degree curve had to be abandoned because of the excessive wear on the wheel flanges. The structure was weak in that it was nailed throughout in timbers which for the most part were dead wood. The posts and sway bracing were poles from which the sap had dried out and this lack of resilience made them weak and brittle. The use of such timber should not be encouraged, even in temporary structures. The revised plans called for the addition of a short conveyer, 250 ft. long on the center line of the B. Conveyer produced, over which the loading hopper was to be built and a Dinky trestle erected along side of it, having a straight alignment.

Except as mentioned above, no other changes were to be made in the conveyer system. On the dam two new chutes were ~~to be~~ built, replacing the old ones which were worn out. These new chutes were ~~to be~~ slightly modified in section, and on approximately the same slope, namely 45 degrees.

During the fall of 1911, a subcontractor built the outlet tower, an illustration of which is shown on page 97 It is of reinforced concrete with a 10 ft. bore and rests on a solid rock foundation. It connects with the main outlet tunnel 150 ft. from the upper portal, by means of another tunnel 10 ft. long. A 20 ft. sump in the bottom of the tower provides against the impact of water falling from the gates above. The three sets of posts, one above the other

and 30 ft. apart, each have 8 openings 2x2.5 ft. in section, which are opposite in pairs. This arrangement destroys impact of incoming water against the inside of the tower.

Two cylindrical castings form the gate for each set after the manner of a cylindrical water wheel. One of them is set in concrete on the inner face of the tower and has the 2x2.5 ft. openings. The other, 3 ft. high, slides down into position just inside the outer stationary casting. Bronze beveled wedges make them fit water tight when the gate is closed. Two stems are provided for each movable casting and the hoisting devices are on top of the tower and operated by hand.

The posts being 30 ft. apart, provide for no gates being operated under more than a 40 ft. head. In the construction of the tower proper, the inside form consisted of two semicircular steel plates, 5 ft. high. For the outside face with its batter of 1 in 48, wooden vertical segments, 5 ft. high, properly tapered and banded together, were used. As the work advanced, a segment was taken out for each 5 ft. rise. A temporary wooden, 4 post tower, built along side of the permanent one carried the guides in which the concrete skip was operated, also the traveler which hoisted the castings and set them in place. An inclined suspension bridge from the tower to the cliff nearby, provides access to the hoisting devices.

That winter the Company installed the big sluce gates at the upper portal of the outlet tunnel. They are 2 in number, set on an incline of 45 degrees, are 5 ft. apart and have a section of 2.5x10 ft. They are made of cast iron,

heavily ribbed and move on bronze slides. When closing, they are drawn tight by bronze wedges, 3 on each side of each gate. The hoisting devices are similar to those used at the outlet tower but designed to operate under a 136 ft. head. The original plans did not include the tower gates arrangement, but after its erection the maximum working head on the big sluce gates became 55 ft. The steel gate stems are 4½ inches in diameter and 190 ft. long, being supported every 17 ft. by torsion guides set on concrete piers.

The designs called for a 3/8" steel plate lining in the tunnel for 20 ft. back from the gates at the portal. This had been installed,when heavy floods came and destroyed the cofferdams. A week later, when construction was resumed,it was found that due to the high water pressures the steel lining had become badly warped and bulged away from the concrete lining proper. Consequently, to provide against any future pressure of reservoir water which penetrates the concrete and lodges between it and the steel lining, 1¼ inch anchor bolts 4 ft. long were added, they being spaced on 18 inch centers both ways, on all the steel linings. As a further precaution, ¼ inch weeper holes were bored in the steel lining just below each bolt head to provide an outlet for the water. Water pressure of this nature had not been anticipated in the original designs, but the existence of any lining in the first place does not seem justifiable, considering the fact that it is impact of water against concrete, rather than friction, which unravels and destroys the latter.

The preliminary work for the season of 1912 began

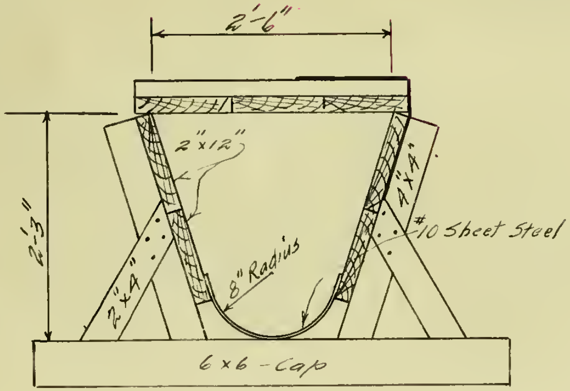

2'-6"
2'-3"
2"x12"
4"x4"
2"x4"
#10 Sheet Steel
8" Radius
6 x 6 - Cap

FIG. 1

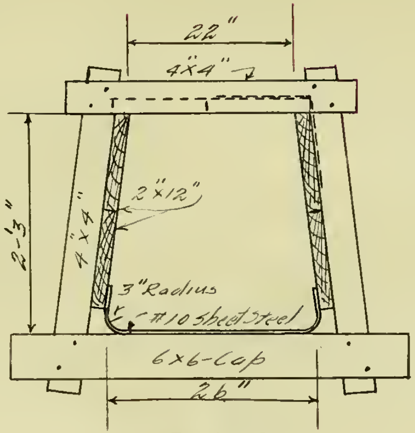

22"
4"x4"
2'-3"
4"x4"
2"x12"
3" Radius
r
#10 Sheet Steel
6x6-Cap
26"

FIG. 2

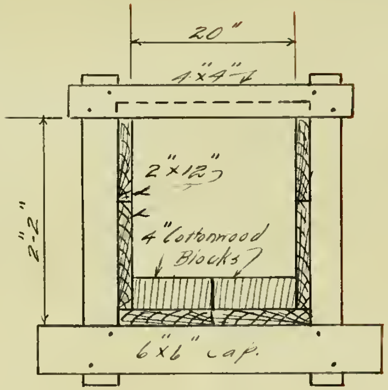

20"
4"x4"
2'-2"
2"x12"
4" Cottonwood Blocks
6"x6" Cap.

FIG. 3

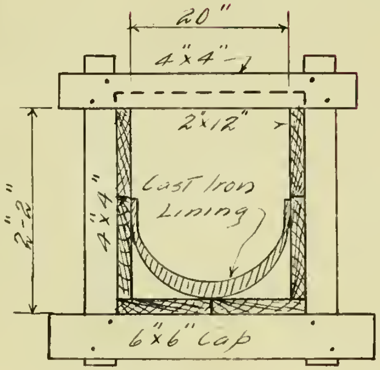

20"
4"x4"
2"x12"
2'-2"
4"x4"
Cast Iron Lining
6"x6" Cap

FIG. 4

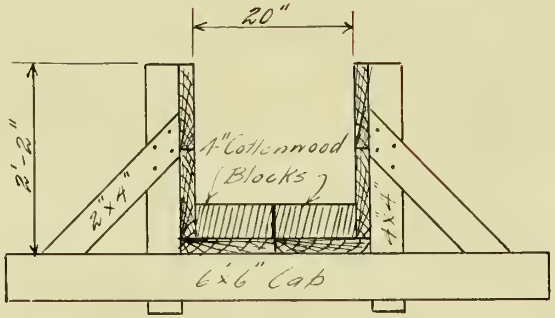

20"
2'-2"
2"x4"
4" Cottonwood (Blocks)
4"x4"
6"x6" Cap

FIG. 5

SECTIONS
OF
EARTH CHUTES & SLUICE FLUME
USED ON
CONSTRUCTION OF THE DAM
Scale 1" = 2'-0"

early. The through cut to the new pit was excavated by steam shovel, working from the top in sections about 10 ft. deep and half the width of the cut, first on one side, then coming back and taking a slice on the other and 5 ft. lower. The loading track was thrown likewise, from one side of the cut to the other. This method had to be used on account of the limited swing of the shovel boom (190 degrees) which would not allow the entire cut to be excavated from one face. Thirty thousand yards were taken out here, the maximum cut being 65 ft. The lower illustration page 105, also pages 93 and 89, show the extent of the changes made at the lower end of the B. Conveyer. A third unit of 250 ft. on a 3.5% grade was added to the System, using a 5 ply Gandy Belt. All electrical installation was changed from D.C. to A. C. machines, vis., a 40 H.P., 2200 Volt, 60 Cycle, 3 Phase type F.H., Westinghouse induction motor for Conveyer A., a 75 H.P. motor of the same type for Conveyer B, and a 30 H.P. motor, also of the same type, for Conveyer C. A 22000 volt transmission line, 3¾ miles long, was built from Oakley to the dam by the Company, current having been contracted for with a Snake River Hydro Electric Company. Two 135 K.W. G. E. Co.'s Air Blast transformers were used and the total line loss was 400 volts.

On the dam 2 new chutes were built having sections as shown by figures 3 and 4, page 109, and with slopes of 47½ degrees, this being the dumping angle of Western Dump Cars.

Excavation in the pit began about April 1st, with the following equipment: 2 - #60 Marion Steam Shovels, 3 - 16T Davenport Dinkies and 50 - 4 yd. Western Dump Cars. This

allowed 2 trains (15 cars each) to be loading while the third was in transit. The material was found to be nearly as hard to excavate as that in the 1911 pit, although Shovel #1 on the lower bench did not require any shooting of its bank. Even with considerable shooting for #2 Shovel, its dipper teeth had to be changed every 6-10 days. Panama teeth, with removable points of manganese steel were used very successfully. Shooting the bank cost on an average 3.5¢ per yd. and churn drilling from 20-27 cents per. lin. ft. Coyote holes were not used much on account of the low banks. Fourty pound rail was substituted throughout, the lighter rails used in 1911 having been very expensive to maintain in serviceable order.

Increasing the slope of the bars on the Loading Hopper from 6 to 1, to 4 to 1, was found to help the material considerably in sifting through them. The conveyers were started in succession, beginning with the A. motor, a signal system being used by the motor men to communicate with each other. In this way, the starting torque did not exceed 150 K.W. Running under no load the system required about 50 K.W. and when under full load, from 100-125 K.W. This variation was caused chiefly by the run of pit material, which at times, had a great deal of gravel in it and again might be all earth.

It was soon found that the Gandy belt of the C. Conveyer was proving unsatisfactory. This type of belt is very sensitive to moisture, and when the wet material was being conveyed, began to stretch. Over 50 lin. ft. were taken out for this cause, which so weakened the belt that it began to unravel and could not be held to the drive drum. A rubber belt

similar to the ones used on the other units, was then substituted. Another type of idler was used on the C. Conveyer, as there was not the underneath clearance available, which is necessary for a gravity idler. The idler used here was placed in a fold of the return belt, which lapped back around a return carrier roller, and was operated in a slotted frame, the slack in the belt being taken up by increasing the width of the fold.

The belts were run on a speed of 400-450 ft. per minute and easily conveyed 3500 yds. in a 10 hr. shift. This amount could have been increased 25% by speeding up the belts and loading heavier. In ordinary material a #60 Marion Shovel should dig from 60,000 to 90,000 yds. per month (both shifts) which is not much less than the capacity of 2 shovels in the 1912 pit. The maximum output for both shifts was made in Sept. 1912, when 105,000 yds. measured in place were conveyed. An average for the season was about 95,000 yds. per month with the night shift doing 90% as much as the day crew. An inspection of the conveyers after the seasons run showed that the belts were worn considerably in places, but would be able to convey the 35,000 yds. remaining to be placed in 1913. There was also considerable pitting of the surface of these belts, because they did not clean themselves entirely of coarse sand, small pebbles, etc., but carried it to the inside drive drum idlers. This could have been avoided by the use of stiff brushes rotating against the belt as it left the first drum. The carrier and troughing rollers were also worn very thin and would not outlast the belt any length of time. Consequent-

ly in figuring dirt costs, the conveyer system must be charged entirely against the Oakley dam of 1,079,000 yds.

The cementing quality of the earth caused considerable choking in the chutes, especially during rainy weather. Both types, figures 3 and 4, page 109, would gradually cake up on the sides and bottom, until a large rock became lodged and prevented the finer material from escaping. Only one chute being operated at a time, made it possible to alternate and clean one while the other was being used. Because of this caking, which continued to some extent through the entire season, the linings of these chutes did not wear preceptibly. The variation of the slope of the chutes where the material was running free, did not materially change their efficiency, the 1$\frac{1}{4}$ to 1 slope operating apparently as satisfactorily as the steeper 47$\frac{1}{2}$ degree slope. Also when the chutes were run full of material with a shut-off gate at its lower end, it was found that even a vertical chute would clog at times. This was due mainly to the small section which these chutes had and it is very probable that larger chutes would also have had a material advantage in the other method of running them partly full.

During the season several changes were made in the methods used in building the core wall. First - 10# rails were imbedded vertically into the core and 8 ft. apart. Instead of using short braces from the earth up to the top of the form to keep it in place, the forms were wired to the rail. Considerable time was saved by this method, it more than offsetting the extra cost of rail. Furthermore, the puddling of

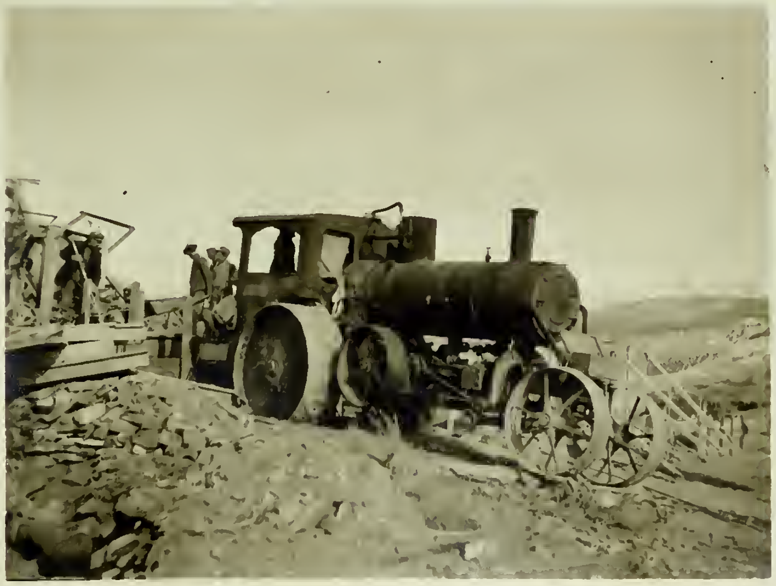
material against the completed core could be kept up more easily. Second - a Smith $\frac{1}{2}$ yd. concrete mixer was mounted on a wagon bed together with a $7\frac{1}{2}$ H.P. motor which did away with all hand mixing. Being a small wall, considerable time was taken up in moving this plant, erecting platforms, runways, etc, to load the mixer from. The writer designed a self charging skip, operated by the same motor that ran the mixer, which did away with all platforms, etc. and reduced the time required in moving the plant from 3 hrs. to 20 minutes. See illustration on page 102. The table below gives the cost per yd. of this core wall, based on an average months run during the season of 1912.

COSTS - REINFORCED CONCRETE CORE WALL

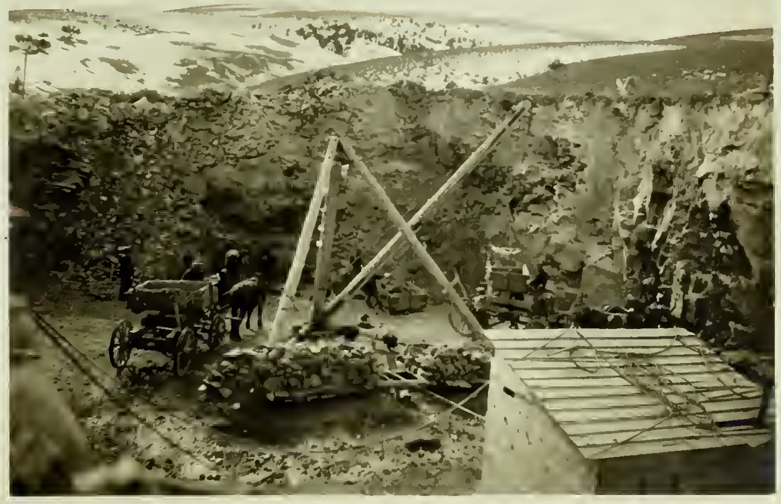
Wire mesh reinforcing ---- 546 yds.

Labor -	Forms, Tying reinforcing wire together, etc.	$ 810.05
"	Mixing (by machine)	638.87
Material -	Cement @ $ 1.00	1959.00
	Reinforcing Wire @ 11.3¢	527.48
	Form Rails @ 4¢	233.20
	Sand @ $ 2.25	873.00
	Crushed rock @ .43¢	83.85
	Mixer	60.00
	Current	50.00
	Lumber, Nails, Wire	15.00
	$	5250.45

$ 9.62 per yd.



AVERY TRACTION ROLLER ENGINE.



STIFF LEGGED DERRICK.
(Spillway Excavation)

In the spillway a stiff legged derrick and electric hoist were installed. The old method of loading dump wagons was rather slow and also hard work. Skips were made which could be loaded on the ground and then raised by the derrick to an especially built carriage on the running gears of a wagon. They were then hauled to the slopes of the dam and dumped much as a Western dump car does. On the 3 to 1 side, these skips were hauled down the slope and the rock dumped in place,while on the other side, steel lined chutes were used when the rip rap was at a considerable distance from the top of the slope. Excavation in the spillway and placing the rock on the slopes as rip rap, cost $1.70 per yard. See illustration page 115 which shows the installation in the spillway.

The following method was used in blasting. The spillway has an average width of 50 ft. and varies in depth from 8 to 26 ft. A breast of holes (usually 5 across the cut) were drilled down to 1 ft. below grade and then sprung 3 times with dynamite, so as to form a pocket for the black powder which was used for the blasting proper. As many as 30 - 25 lb. cans of powder could then be charged in a single hole.

As stated before, black powder acts much more slowly than dynamite and in this case tended to lift the rock out of place as well as break it up. The specifications for rip rap called for rock as near 1 cu. ft. in size as possible. As the rock originally had a great many crevices, the use of dynamite would have broken it up so small as to be unfit for further use.

When completed, a concrete lip will be built on the lower side of the spillway channel, south of the core wall and 7.5 below the crest of the dam, thereby providing 600 lin. ft. of waste weir with a capacity of 2000 sec. ft. for a 1 ft. depth of over-flow.

The outlet gates were entirely closed in March 1913, and all rock which had fallen from the roof of the tunnel was removed. The gates were found to work very satisfactory with no leakage; but beyond the lined portion of the tunnel, streams of water, estimated as 4 sec. ft., escaped through the earth seams. The water naturally washed the earth out, causing the roof to cave in. It would appear that such conditions as these would require that the entire tunnel be lined.

A change was made in the up-stream slope of the dam above the water line. Instead of continuing the 3 to 1 slope to the crest, it was changed to 2 to 1 beginning 7.5 ft. below the top of the dam. This, not only will make the rip rap better able to deflect any wave action, but also make the placing of earth in the fill much easier for the contractor.

The following tables give the cost per yd. of placing the earth in the embankment. In Table 1 under the head Depreciation, interest on principle expended, repairs, and deterioration are included. The method of computation used was based largely on those employed by the U. S. Reclamation Service at the Minidoka and Boise-Payette Projects.

UNIT COSTS - EARTH FOR OAKLEY DAM.

TABLE 1.

Principle Equipment Used.

	Cost	Depreciation.
2 - #60 Marion Steam Shovels	$ 14000	$ 7700
3 - 16T Davenport Locomotives	5400	2430
50- Dump Cars (4 yd.)	7500	4875
1 Mile Track, 40 lb. rail	2500	1000
Conveyors (Belts,Housing,Trestles,Hoppers)	33000	31350
3 - Motors (conveyors)	3800	380
3.75 Miles Transmission Line	4000	400
1 - Motor (sprinkling system)	750	75
Pipe Line (sprinkling system)	750	750
Traction Roller Engine	2000	1200
	$ 73700	$ 50160
		4.6¢ per yd.

TABLE 2.

Excavation by Steam Shovel	6.7¢
Hauling, track maintenance, etc.	2.2¢
Conveying, unloading at Hopper	3.9¢
Distribution on Dam	6.8¢
Sprinkling & Rolling	0.6¢
Engineering Supt., & Gen'l. Expense	3.9¢
Depreciation (see above)	4.6¢
Cost per yd. in Embankment	28.7¢

CONCLUSION.

At the present time, March 15, 1913, construction on the Twin Falls Oakley Project is in the following state of completion: Dam 97%, Outlet Tunnel, Gates and Tower 100%, Spillway 95%, Feeder Canals 95%, and Distribution System 97%. By June 1st, the Company will have finished its entire contract here.

In April, 1912, the Outlet Gates were closed and the storing of water begun, at the same time supplying the local settlers with the amount of water granted by their old water rights. There are now some 17,000 acre ft. of water in the reservoir which should be increased 100% within the next 30 days, as the spring run off has hardly begun. This amount will then be more than enough to supply all settlers for the first year in the operation of this system.

During the season of 1912 measurements were taken of all water entering the reservoir, and by comparing it to that discharged for irrigation, it was found that the storage losses in the reservoir were remarkably low, being 5%.

In the chapter on Preliminary, it was shown that the assumed run off from the drainage basin was very conservative as compared to the results obtained from computations, as well as by comparison to those of other tracts similar and adjacent to this one. Immediately after construction was begun, the

U. S. Geological Survey installed automatic measuring stations on all four creeks which supply the Oakley reservoir and these were regularly attended by local hydrographers in the employ of the Twin Falls Land and Water Co., owners of the project. The records give the following total run off for the year 1912, vis., Goose Creek 56,771 Acre Ft., Trapper Creek 11,373 Acre Ft. Cottonwood Creek 13,705 Acre Ft., and Birch Creek 6,643 Acre Ft., or an aggregate total of 88,492 Acre Ft.

Deducting the amount of water held by water rights of settlers at Cottonwood and Birch Creeks, also the storage and transportation losses of the irrigation system, it is seen that the above run off is less than the amount necessary for the entire segregation.

On the other hand, too much reliance should not be placed on the above figures for the following reason; the percent of the total annual run off which comes down these creeks in one month during the spring floods will vary from 28 to 34%, which often causes the streams to overflow their banks and make the records of these gauges worthless at a time when they should be most accurate. No stream measurements can be considered as having much less than 5% error when measured by these gauges and on an average, this error will be nearer 10%. Added to this, one must consider the personal reliability and capableness of the observer who operates these instruments. The extreme conservativeness of the original estimates and computations, as well as the favorable comparison of this drainage area with other adjacent ones whose run offs are known, tend to discredit any fear that there will be a shortage

in the available water supply here.

A duty of water of $1\frac{1}{2}$ acre ft. delivered on the land, should be sufficient, in view of the opinions expressed by settlers who have farmed in this valley for the past 20-30 years. However, instead of limiting the duty of water to any fixed amount numerically, it would be well to make some provision that would consider not only the increasing scarcity of water for irrigation purposes, but also protect the farmer, that he may have enough to successfully irrigate his land. To do this, the term "beneficial use of water" should appear in all contracts and the State, the original owners of the water, should determine from time to time, what this unit for beneficial use is.

All water used for irrigation will be accurately measured in the canals just below the main head gates. The canals will be lined for 100 ft. with concrete and flow measurements taken with a current meter. It is now understood that the Water Company will not use the lateral measuring gates to determine the amount of water supplied to each settler, but will install Cippoletti Weirs for this purpose. A bad feature of a pipe outlet is that a considerable head is necessary in the Canal before a sufficient area of the pipe section is reached to give a satisfactory discharge into the lateral. This refers expecially at the lower ends of the canals when the water required for this head is not all needed for irrigation and consequently is wasted.

In the proposed distribution of water to the settlers by rotation, only one main canal will be in operation at a time. For example, settlers on the East Side Main Canal will be

notified that they will have water for such and such a period. Water will then be turned into this canal and ditch riders will open the farmers' gates on all laterals of this ditch. Under this system all ditches will run full, which means the maximum available head at each lateral gate, thereby insuring a rapid delivery to each farmer at a minimum transportation loss. When the required amount of water has been delivered, the gates will be closed and the same operations repeated on another canal. About 15 ditch riders will be needed on this project, and they will use motor cycles in making their rounds to open and close the gates and take observations on weir gauges.

Circulars have been issued by the Water Company announcing the availability of water for the season of 1913, and within the next 6 months, nearly 500 more families will have joined the "back to the farm movement" with every prospect for a prosperous and independent future.

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